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# *GROUND MOTION AMPLIFICATION AND SEISMIC LIQUEFACTION: A STUDY OF TREASURE ISLAND AND THE LOMA PRIETA EARTHQUAKE*

***Abstract*** This report is part of the Navy's Earthquake Hazard Mitigation Program. It describes the effects of the Loma Prieta earthquake on Naval Station, Treasure Island focusing on geotechnical comparison of performance of marginal and improved sites. Procedures are presented to estimate settlements and results are compared to observed data. A detailed analysis of ground motion is presented in conjunction with site amplification. It is shown that the site amplification at Naval Station Treasure Island is caused by the stiff Bay Mud layer rather than the loose sand deposits. A comparison is made to the Mexico City earthquake of 1985, which also had high plasticity stiff clays. The high stiffness with shear strain exhibited by these deposits in comparison with normal clays is shown as a function of high plasticity and should be a warning to engineers of potential site amplification of ground motion from distant earthquakes.



NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME CALIFORNIA 93043-5003

# METRIC CONVERSION FACTORS

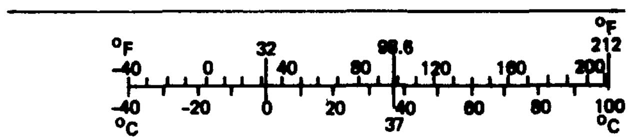
### Approximate Conversions to Metric Measures

| Symbol | When You Know | Multiply by | To Find | Symbol |
|---|---|---|---|---|
| | | **LENGTH** | | |
| in | inches | *2.5 | centimeters | cm |
| ft | feet | 30 | centimeters | cm |
| yd | yards | 0.9 | meters | m |
| mi | miles | 1.6 | kilometers | km |
| | | **AREA** | | |
| $in^2$ | square inches | 6.5 | square centimeters | $cm^2$ |
| $ft^2$ | square feet | 0.09 | square meters | $m^2$ |
| $yd^2$ | square yards | 0.8 | square meters | $m^2$ |
| $mi^2$ | square miles | 2.6 | square kilometers | $km^2$ |
| | acres | 0.4 | hectares | ha |
| | | **MASS (weight)** | | |
| oz | ounces | 28 | grams | g |
| lb | pounds | 0.45 | kilograms | kg |
| | short tons (2,000 lb) | 0.9 | tonnes | t |
| | | **VOLUME** | | |
| tsp | teaspoons | 5 | milliliters | ml |
| Tbsp | tablespoons | 15 | milliliters | ml |
| fl oz | fluid ounces | 30 | milliliters | ml |
| c | cups | 0.24 | liters | l |
| pt | pints | 0.47 | liters | l |
| qt | quarts | 0.95 | liters | l |
| gal | gallons | 3.8 | liters | l |
| $ft^3$ | cubic feet | 0.03 | cubic meters | $m^3$ |
| $yd^3$ | cubic yards | 0.76 | cubic meters | $m^3$ |
| | | **TEMPERATURE (exact)** | | |
| °F | Fahrenheit temperature | 5/9 (after subtracting 32) | Celsius temperature | °C |

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.


### Approximate Conversions from Metric Measures

| Symbol | When You Know | Multiply by | To Find | Symbol |
|---|---|---|---|---|
| | | **LENGTH** | | |
| mm | millimeters | 0.04 | inches | in |
| cm | centimeters | 0.4 | inches | in |
| m | meters | 3.3 | feet | ft |
| m | meters | 1.1 | yards | yd |
| km | kilometers | 0.6 | miles | mi |
| | | **AREA** | | |
| $cm^2$ | square centimeters | 0.16 | square inches | $in^2$ |
| $m^2$ | square meters | 1.2 | square yards | $yd^2$ |
| $km^2$ | square kilometers | 0.4 | square miles | $mi^2$ |
| ha | hectares (10,000 $m^2$) | 2.5 | acres | |
| | | **MASS (weight)** | | |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.2 | pounds | lb |
| t | tonnes (1,000 kg) | 1.1 | short tons | |
| | | **VOLUME** | | |
| ml | milliliters | 0.03 | fluid ounces | fl oz |
| l | liters | 2.1 | pints | pt |
| l | liters | 1.06 | quarts | qt |
| l | liters | 0.26 | gallons | gal |
| $m^3$ | cubic meters | 35 | cubic feet | $ft^3$ |
| $m^3$ | cubic meters | 1.3 | cubic yards | $yd^3$ |
| | | **TEMPERATURE (exact)** | | |
| °C | Celsius temperature | 9/5 (then add 32) | Fahrenheit temperature | °F |



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This report is part of the Navy's Earthquake Hazard Mitigation Program. It describes the effects of the Loma Prieta earthquake on Naval Station, Treasure Island focusing on geotechnical comparison of performance of marginal and improved sites. Procedures are presented to estimate settlements and results are compared to observed data. A detailed analysis of ground motion is presented in conjunction with site amplification. It is shown that the site amplification at Naval Station Treasure Island is caused by the stiff Bay Mud layer rather than the loose sand deposits. A comparison is made to the Mexico City earthquake of 1985, which also had high plasticity stiff clays. The high stiffness with shear strain exhibited by these deposits in comparison with normal clays is shown as a function of high plasticity and should be a warning to engineers of potential site amplification of ground motion from distant earthquakes.

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## INTRODUCTION

The Loma Prieta Earthquake of October 17, 1989 caused $125 million dollars of damage to U.S. Navy facilities. The predominant cause of damage was liquefaction of cohesionless waterfront deposits. This is a major continuing problem faced by the Navy which because of mission requirements must be situated at the waterfront often on marginal soils. This report will analyze the response of soils at Naval Station, Treasure Island documenting settlements observed and serving as a benchmark to calibrate soil settlement prediction algorithms. Of significance to this event was the fact that 80 percent of the loss of life occurred in a heavily concentrated damaged area approximately 50 miles from the fault rupture zone. The local site soil effects are the most striking feature of the Loma Prieta event. Reference 1 through 20 were used as part of this study.

### The Earthquake

The earthquake (References 1,2,3) occurred when a segment of the San Andreas fault northeast of Santa Cruz, California ruptured over a length of 28 miles producing a Richter local magnitude, $M_L$, of 7.0 and an average surface wave magnitude, $M_S$, of 7.1. The epicenter was 10 miles northeast of Santa Cruz and 20 miles south of San Jose. Figure 1 shows the fault rupture main shock and the numerous after shocks in both plan view and in two section views. Figure 2 illustrates a cross-section of the fault region showing the dipping fault plane and the hypocenter, Reference 2. The initial rupture length was estimated to be 24 miles. The main rupture began at a depth of 11 miles below the earth's surface and near the center of what would be the rupture plane. Over the next 7 to 10 seconds the rupture spread approximately 12 miles to the north and 12 miles to the south The unusual middle location of the hypocenter within the rupture location contributed to the unusually short duration of the event. Approximately 8 to 10 seconds of strong shaking was observed which is considerably less than would be expected from an event of this size. The rupture propagated towards the earth's surface but during the main event appears to have stopped at a depth of 3 to 4 miles.

The section of fault rupture for the Loma Prieta earthquake is the southern part of the section of the San Andreas which ruptured in the 1906 magnitude 8+ earthquake. This part of the fault contains a bend resulting in a localized compression zone. Figure 2 illustrates the fault showing the right lateral (horizontal) and reverse ship (vertical) motion. The right lateral offset was 6.2 feet and the vertical offset was 4.3 feet, Reference 1. This combination was a significant cause of the unusually high vertical ground surface accelerations. Figures 3 and 4 show the ratio of vertical acceleration to horizontal acceleration. In Figure 3 both components are considered and in Figure 4 the average horizontal component is used. These figures show that in the region near faulting vertical components exceed the traditional level of 50 percent of horizontal components. A total of 51 aftershocks of magnitude 3.0 or larger occurred within the first 24 hours after the main shock. Figure 5 shows the region affected by the event. Several factors combined to keep damage and loss of life down:

The rupture occurred in a sparsely populated location.

The duration of strong motion was short.

The event hypocenter was centrally located and spread uniformly.

## NAVAL STATION TREASURE ISLAND

### Geology

Figure 6 shows the faulting around the Treasure Island site and the Loma Prieta earthquake rupture zone (References 4,5 and 6). Treasure Island is a man-made island constructed in the 1930s and situated between San Francisco and Oakland and attached to Yerba Buena Island by a short causeway, Figure 7, Reference 7. Over 29 million cubic yards of mostly fine-to-medium grained sand was dredged from borrow areas in the San Francisco bay and used as fill material over the Yerba Buena Shoals north of Yerba Buena Island. The bottom of the shoals area varied in depth from -2 feet to - 26 feet mean lower low water. About 65 percent of the bottom sediments in this area were composed of sand and the remainder was soft clay. A low mound of rock was placed along the perimeter of island to act as a retaining dike for the sand fill, Figure 8. The fill material was deposited hydraulically by using a pipeline, by hopper and by clam shell dredge. Where the depth of the shoals exceeded -6 feet a bed of hydraulic sand fill was placed. The dikes were constructed such that each succeeding level was placed inward of the lower dike and rested on previous levels of hydraulic fill. Fill was placed to a level of +13 feet. Photography of the construction shows that the dike was constructed in segments starting at the southwest corner. A low weir was installed to allow water from the hydraulic dredges and soft mud displaced by the fill to escape. A small dredge was used to remove areas of entrapped mud.

During construction a 500 foot length of the north end of the east perimeter dike settled 10 to 14 feet. This failure area was stabilized by flattening the slope and placing a layer of sand beyond the dike toe. The north dike design was modified by first excavating a 400-foot trench 20 to 30 feet deep and backfilling with coarse sand as a foundation for the rock dike, Reference 7.

**Soil Conditions**

The ground level of Treasure Island varies from +10.5 to +16 feet above mean lower low water level with a water table between +6 feet and +0 feet. Water levels are affected by the permeability of the sand fill and vary with proximity to the perimeter dike. The ground water levels at the center of the island are less affected by the tide and vary from 5 to 8 feet below the ground surface. During the earthquake it is thought the water table near the island perimeter was at +3 feet. Numerous explorations have been made by drill and sample or cone penetration testing, Figures 9 and 10, References 4, 5 and 6. Subsurface materials can be divided into four strata:

1) Loose to medium dense hydraulically placed sand fill;

2) Loose to medium dense native material, Yerba Buena Shoals sands and medium stiff native clays,

3) Recent Bay sediments of medium stiff olive gray silty clay (Bay Mud) but containing some soft clay zones; and,

4) Older Bay mud sediments consisting of brownish and greenish gray very stiff sandy, silty and/or peaty clays and dense sands.

The fill material is a fine to medium grained sand and has gradations ranging from well to poorly graded. It contains varying amounts of gravel, silt and clay. In general the fill material is of lower density, looser, has a lower penetration resistance and lower shell content than the native shoals sands and clays. However from an engineering perspective the fill and native materials are thought sufficiently the same from an earthquake performance basis so as to be classified as a common strata. Thickness contours of the fill and native shoals strata are shown in Figure 11 and is seen to vary between 30 to 50 feet for the most part. Figure 12 gives cross sections of Standard Penetration Test corrected blowcounts for this strata. The blowcount data is reported from recent data using appropriate penetration procedures (rotary wash drilling techniques, appropriate hammer size and drop etc.) and from Cone Penetrometer Test probes from which data was converted to blowcounts utilizing site specific correlations. Corrected data from previous investigations has also been included. The blowcounts show the material to be loose to medium dense and susceptible to liquefaction under seismic shaking. Note the extensive shaded region containing blowcounts less than 10. Values less then 10 would have a high probability of liquefaction under moderate earthquakes and are high hazard regions regardless of what procedure of analysis is used.

Below the fill and native material layer is a layer of Bay Mud composed of a medium plastic silty clay with interbedded regions of sand and silt. These recent Bay sediments vary in thickness between 10 and 120 feet. The layers were deposited in a marine environment and are normally consolidated.

The older Bay sediments of Pleistocene age are generally stiff to sandy, silty and or peaty clays that extend down to the Franciscan bedrock. The layer is lightly to moderately overconsolidated. Bedrock is at a depth of about 280 feet below ground surface. Thickness of the older Bay sediments is estimated to range from 20 to 170 feet.

Figure 13 shows a plan view of Treasure Island where post earthquake testing was performed by Hryciw et. al., Reference 5. Based on seismic cone penetration test shear wave propagation velocities for the fill materials and the recent Bay Muds were determined, Figures 14 and 15. Equations of best fit were computed. Shear modulus and damping as functions of shear strain were determined for use in analysis, Figure 16.

## Ground Motion

Strong motion recordings were obtained from an instrument on Treasure Island, Figure 17, References 1, 2 and 5. The peak horizontal ground acceleration components from the main shock were 0.16g and 0.10g. A significant factor in the Loma Prieta earthquake was the amplification of ground motion in areas underlain be thick deposits of Bay sediments. Treasure Island falls within this observation especially in comparison with recordings on nearby Yerba Buena where the peak horizontal acceleration recorded on a rock site were about three times less than those on Treasure Island. This will be discussed in the following section. Observation of the Treasure Island record shows that at about 15 seconds after the start of recording, the ground motion was subdued; this was probably caused by the occurrence of subsurface liquefaction. Liquefaction occurred after about 4 or 5 "cycles" of shaking after about 5 seconds of strong motion. Sand boils were observed at numerous locations and bayward lateral spreading occurred with associated settlements. Ground cracking was visible with individual cracks as wide as 6 inches. Overall lateral spreading of 1 foot was estimated. Ground survey measurements indicate that settlements of 2 to 6 inches occurred variably across the island and that some areas had as much as 10 to 12 inches of settlement. The liquefaction related deformations resulted in damage to several structures and numerous broken underground utility lines.

## Yerba Buena

Yerba Buena Island is a large rocky outcrop. Figure 18 shows the accelerogram components recorded for the main shock. Note that the horizontal components were 0.068g and 0.031g, both significantly less than those on Treasure Island. Figures 19 and 20 show the 5 percent damped response spectra for the two sites for both horizontal components of motion illustrating the spectral amplification. Note the significant soft soil amplification shown in Figure 19 for Treasure Island and compare it to Figure 20, Reference 2.

## EARTHQUAKE DAMAGE

### General Ground Deformation

There was extensive evidence of settlement and differential settlement adjacent to or inside buildings. The region around Buildings 2 and 3 which was constructed on piles settled on the order of 4 to 6 inches while the structures remained at their original elevation. There was evidence of extensive liquefaction by the presence of sand boils up to 20 feet in diameter. The large sand boils with the accompanying settlement and expulsion of water created areas of ponded water.

Figure 13 shows locations of test performed by Hryciw (Reference 5). At the UM1 location no damage was seen. Some liquefaction occurred in adjacent inland areas near UM3 and subsidence of the retaining dike was noted at UM12. During construction the north eastern section of the dike settled extensively and the dike design was modified. At this location, a 400 foot long trench was dug to a depth of of 20 to 30 feet and was backfilled with sand.

Lateral spreading and settlement of the dike occurred in a number of locations. The maximum settlement at the top of the dike was nearly 2 feet. Lateral spreading cracking was widely observed. It is estimated that 8 inches of the settlement was caused by shaking-induced compaction and the remainder was lateral spreading. A 2500 foot long crack opened near the east side of the island and passed through Building 7. The floor slab cracked and a major sand inflow occurred filling the ground floor of the building with sand to a depth of 6 inches in one area. The area between 3rd and 5th Streets and near 11th Street exhibited lateral spreading movements having cracks of 0.5 inches. Summation of horizontal crack widths indicates a total bayward movement of 6 to 12 inches may have occurred at the dike area. Survey data shows that at Avenue N and 3rd Avenue approximately 10 inches of lateral movement occurred primarily eastward. Most spreading movement was restricted to an area within a distance of 200 feet from the perimeter dike. The lateral spreading occurred only during the period of actual earthquake motion and did not continue to move afterward.

Ground surface settlements of up to 12 inches occurred. Examination of subsurface conditions indicates that the liquefied zones may have extended to within about 5 feet of the ground surface. Six buildings suffered significant damage. Several structures of the 1930's vintage suffered cracking, separation of floor slabs and minor sand inflows. More than 40 underground utility breaks occurred. Pre- and post-earthquake survey data were used to construct Figure 21 showing the shaking induced compaction settlements as reported by Egan et. al, References 4 and 8. They used blowcount data to estimate settlements to expand the limited settlement survey data. They initially used a procedure by Tokimatsu and Seed (Reference 14) which was found to over estimate settlements by a factor of two. A modified procedure was developed which incorporated the liquefaction assessment models of Liao and others (Reference 15) and the influence of grain size on post-cyclic volumetric strain after Lee and Albaisa (Reference 16). Figure 21 shows elevation changes at survey monuments or reference points. There was good agreement between the estimated settlements and observed survey settlement data. Figure 21 shows that for the thickest zones of liquefied soil, settlements of 8 to 10 inches may have occurred.

Most buildings on Treasure Island suffered no or minor damage which was limited to minor cracking or differential settlement. Several buildings near the perimeter dike or areas of significant lateral spreading did experience greater damage. These areas experienced 6 to 10 inches of settlement. Buildings which were situated in areas which settled less than 6 inches generally experienced only minor damage. Figure 22 summarizes damage.

Forty-four breaks in utility pipelines included 28 fire and freshwater lines of steel or asbestos cement, 10 sewage lines of vitrified clay and 6 welded-steel gas lines. Many of the breaks occurred near the dike in areas of high lateral spreading. Crude estimates of lateral spreading required to cause failure are:

| Type | Pipe Diameter | Spreading to Induce Failure |
|---|---|---|
| Steel or Asbestos Cement | 0 to 4 in | 1 inch |
| Steel or Asbestos Cement | 12 to 16 in | 6 to 12 inches |
| vitrified clay pipe | | 1/4 inch |

### Densified Soil

The area by the wharf at the south-eastern corner of the island was densified by Terraprobe. It showed no signs of liquefaction, spreading or settlement. In areas adjacent to the improved area major sand boils were observed so it may reasonably be concluded that the site improvement kept the area from liquefying. Lateral spreading was not observed adjacent to the north section of the dike which was modified during construction and placed on a foundation of sand, Reference 13. In this area the dike subsided more than in adjacent areas.

## IMPROVED SITE RESPONSE

### Medical Dental Building

The Medical/Dental building was under construction at the time of the Loma Prieta earthquake. The subsurface section along the east-west centerline of the project site is shown in Figure 23. Note that the soil to a depth of 31 to 43 feet is loose to medium dense hydraulically placed sand fill, generally fine to medium grained containing less than 10 percent material finer than the #200 sieve. The zone contains occasional thin layers of soft compressible silt. Beneath this layer is a soft to medium stiff clayey silt (Bay Mud) layer of 30 foot depth. The Bay Mud is underlain by alternating layers of dense to very dense sands and stiff to hard clays, Reference 13.

The structure was to consist of a two story steel frame structure. Typical column spacing was 30 feet and the dead plus live column loads are up to 230 kips. The site is potentially liquefiable. The structure could have been supported on piles or spread footings with site densification. It was decided to densify the upper layer of sand fill to a minimum relative density of 75 percent beneath the building extending to a distance 20 feet beyond the building. Vibro-replacement technique using gravel backfill was chosen as the method for densification. Tests were conducted to determine probe spacing. It was decided to use 10 foot probe spacing to a depth of 22 feet. Since there were zones of silt and clay in the sand fill, the specifications did not require improvement in areas where the Cone Penetration Test friction ratio was greater than 2 percent. The uppermost few feet were to be compacted to 95 percent relative compaction using surface compactors. A study of pre and post densification penetration test showed that the upper 10 feet of sand was already dense and was not improved. The lower level of the layer between depths of 10 to 22 feet which was initially loose was densified. Using a volume calculation method it was estimated that the upper 22 feet were densified to a relative density range between 77 and 80. The silty sand fill interbedded with zones of silt and clay which underlay the upper zone to a depth of 40 feet was not adequately densified; no attempt was made to densify soils in the 22 to 40 foot range. At the time of the earthquake about 40 percent of the footings were built and two 22-foot deep elevator shafts were excavated.

The bottom 8 feet of the 22-foot deep elevator shafts were filled with sand from liquefaction flows of the untreated sand fill between a depth of 22 to 40 feet. Total settlement of the site could not be determined since the benchmarks settled, differential settlement of the footings of 0.88 inches over a distance of 180 feet was noted. It appears that liquefaction did not occur in the upper 22 feet.

**Building 450**

Building 450 is a three story steel frame office building built in 1967. The structure is actually composed of two buildings, the first 160 feet by 160 feet in plan and the other 54 by 124 feet in plan located 100 feet northwest of the first. The buildings have concrete walls and floors; typical dead plus live column loads are 250 to 300 kips. The subsurface section along the north-south project centerline is shown in Figure 24. The profile is similar to that in Figure 23 with the upper 30 feet being loose to medium dense hydraulically placed sand fill underlain by a layer of soft to medium stiff silty clay approximately 20 feet thick (Bay Mud). Note the sloping bedrock formation, Figure 24. It was decided to use densification since some piles would be end bearing on bedrock and some piles would only rely on friction. Conventional concrete piles were viewed as a high cost alternative lacking lateral stability if liquefaction occurred. Sand compaction piles spaced 4 feet on centers beneath footings and 5 feet on centers beneath floor slabs was chosen as the method for site improvement. The specifications called for a minimum relative density of 75 percent in the sand fill to a depth of 30 feet beneath the footings and 65 percent to a depth of 30 feet beneath the floor area and to a distance of 10 feet beyond the building perimeter. The mandrel used for the sand piles was a 14-inch diameter steel casing fitted with a loose steel bottom plate. The mandrel was driven to the required depth and backfilled with coarse sand. The top end of the mandrel was closed and a minimum air pressure of 100 psi was applied to the column of sand as the mandrel was withdrawn. Figure 25 shows Standard Penetration Test blowcounts before and after densification. While the average relative density exceeded minimum requirements and the average overall densification was adequate, there were isolated zones of silt and clay which were not densified to minimum requirements, Reference 13.

There was no evidence of damage to this structure. Some lateral spreading, sand boils and localized settlement were observed outside the improved area.

**Buildings 487, 488 and 489**

The three story buildings with exterior and interior concrete block walls, precast concrete floor slabs, and concrete slab on grade first floors were constructed in 1973. Typical dead plus live loads for the bearing walls are in the range of 5 to 7 kips per foot. The building does not have columns or independent footings. Figure 26 shows the subsurface section along the east-west centerline of the project site. The upper 24 to 33 feet of soil are very loose to medium dense hydraulically placed sand fill with occasional layers of soft compressible silts and clays. Below this layer is a zone of dense sand underlain by a 4 foot thick layer of soft silty clay (Bay Mud). The structure rests on spread footings bearing on densified sand fill. The specifications required that the sand fill be improved to a relative density of 75 percent to a depth of 30 feet beneath the buildings to a distance of 10 feet beyond each buildings perimeter. Vibrocompaction was used for densification. The specification called for the vibrator to be inserted at each compaction point to a depth of 30 feet below grade and maintained at that depth for a period of one minute and then withdrawn at a rate of not more than 1 foot per minute. Crushed rock was continuously placed around the vibrator during the process. The upper 1 foot of sand was compacted to 95 percent relative compaction using a vibratory compactor. Standard Penetration Tests taken indicate that relative densities equaled or exceeded specification minimums.

Building 487 experienced minor cracking in the concrete floor slab caused by differential settlement. The other buildings did not sustain any damage.

### Buildings 452 and 453

Buildings 452 and 453 consist of six wings four stories high radiating from a central core. Building 453, built in 1969 has its core loads carried by circumferential and radial walls; dead plus live loads are approximately 7 kips per linear foot of wall. Figure 27 shows a subsurface profile along a north-south centerline of the project site. The upper 45 feet consists of loose to medium dense hydraulically placed sand fill underlain by about 20 feet of soft to medium stiff clayey silt (Bay Mud). Below the Bay Mud are alternating layers of stiff clays and dense sands. Figure 27 shows blowcounts and indicates that liquefaction would be expected as evidenced by blowcounts below 10. The building specification called for densification to 70 percent relative density to a depth of 30 feet under the building extending 10 feet beyond the building perimeter. The upper 4 feet were to be excavated, backfilled and compacted to a minimum of 95 percent relative compaction. Nonstructural timber piles were used as the means for densification of both buildings. The piles were 8 inches in diameter at the tip and 12 inches at the butt, 20 feet long and driven 25 feet into the fill. The tops were driven to a depth below the water table to prevent deterioration. Unfortunately data giving post-driving density is not available, Reference 13.

After the earthquake the buildings were inspected and no major structural damage was observed. At Building 453 there was one concrete spall and cracks in the floor system. Some repairs were required for the slab on grade which had settled less than 3/8 inch. No significant damage to Building 452 was observed. Sand boils or foundation distress were not observed around the building.

## SOFT SITE RESPONSE

### Treasure Island Site Parameters

Hryciw et. al., Reference 5, performed post-earthquake cone penetration tests to determine shear wave velocities. This data was presented above, Figure 14 and 15. They estimate that the shear wave velocity in the sand fill upper layer was defined by a best fit of the data as:

$$V_S = 150 + 4\,z$$

where z is depth in meters and $V_s$ is in meters per second.

They estimate the shear wave velocity for the Bay Mud layer to be:

$$V_s = 30\,z^{0.55}$$

For the alternating layers of stiff clay and dense sands the following is estimated:

$$V_s = 250 + 2\,z$$

For the purposes of constructing a site model the following additional information can be used.

| Soil | Depth | Shear Wave Velocity |
|---|---|---|
| Silty sand | 100 to 141 ft. | 1100 ft./sec. |
| Stiff to hard clay | 141 ft. | 1100 ft./sec. |
| Stiff to hard clay | 285 ft. | 1400 ft./sec. |

Hryciw et. al., Reference 5, used the computer program SHAKE to analyze the soil profiles at Treasure Island. SHAKE is a frequency domain analysis using strain dependent elastic material parameters. They ran a parameter analysis to consider the uncertainty in specification of the older bay deposits using the measured rock accelerogram recorded on Yerba Buena as the basis for bedrock motion. Their results which would be classed as typical response of this category of analysis are shown in Figure 28. They compute a peak surface acceleration of 0.18 g in comparison with the recorded value of 0.16 g. The recorded spectra exceeds the computed spectra which is explained by Hryciw that the SHAKE analysis does not take into account the softening of the upper layers with the onset of liquefaction. Their analysis shows that the upper layer most amplified the motion, Figure 29. They correlated increase in computed surface acceleration with regions of observed increase in seismic effects.

Seed et. al., Reference 2, analyze the same site using the following:

| Soil Type | Depth | Shear Wave Velocity |
|---|---|---|
| Loose Sandy Fill | 0 to 30 ft. | 500 to 600 ft./sec. |
| Loose silty sand | 30 to 45 ft. | 550 to 650 ft./sec. |
| Silty clay Bay Mud | 45 to 100 ft. | 500 to 700 ft./sec. |
| Dense sand & silty sand | 100 to 140 ft. | 1100 ft./sec. |
| Stiff to hard clay | 285 ft. | 1100 to 1400 ft./sec. |

The calculated peak ground acceleration of 0.18 g agrees with the measured value of 0.16 g. The results show a spectral amplification of 4 to 5 although the computed spectral content is lower than observed, Figure 30. Seed et. al. explains the underestimation of the computed spectra as caused by surface waves generated by the dipping of the Yerba Buena rock outcrop beneath the alluvium and fill of Treasure Island. The late occurrence of liquefaction would not explain the underestimation of long period spectral response ( > 0.15 sec). They note that peak spectral response occurs at a period of 0.6 seconds and has a secondary response at 1.3 seconds. The 0.6 seconds does not represent the predominant period of this deep soft site but rather would be at the 1.3 seconds. The 0.6 second peak results from the site being strongly excited by the input rock motion having an energy concentration at this period.

One should conclude that the soil column analysis using SHAKE is an effective tool for predicting approximate levels of response. It is a simple expedient tool to use requiring data that is usually easily available. It can reliably indicate amplification when the site is properly modeled. However, it does not account for softening of material properties or sloping bedrock conditions. When applied to the Treasure Island site it reliably warns the engineer that substantial amplification will occur. This warning can be used as a screening indicating additional analysis may be needed when warranted by the construction project. Caution should be exercised in using code provisions which may not be representative of the worst case conditions at a soft site. It would be expected that at high levels of ground shaking significant nonlinear response would occur which would not exhibit the same levels of amplification as at low levels of shaking; however the absolute values of motion could be higher. Seed et. al. (1992) shows results for 3 additional sites where SHAKE was in reasonable agreement with observed spectra, Figure 31. They conclude that nonlinear methods in theory may equal or exceed the accuracy of linear methods; but, in practice the analyses often under-predict results because overdamping or poor modeling negate any advantage.

## LIQUEFACTION PREDICTION

### Current Capabilities

A question of importance to the Navy is whether we can predict with high reliability the locations on a Navy base which will experience liquefaction. In the case of Treasure Island Standard Penetration Blowcounts of less than 10 exist in widespread areas. We have had tools for over 20 years which tell us that areas where blowcounts are less than 10 will liquefy during modest levels of ground shaking. In 1985 Seed et. al. updated data using the Standard Penetration Test. The Standard Penetration Test was standardized to an energy level of 60 percent of the free fall energy of the hammer.

Figure 32 shows Seed's, Reference 21, liquefaction assessment chart where the soil conditions are defined by the Standard Penetration Resistance, $(N_1)_{60}$, and the earthquake loading by average shear stress divided by the effective vertical confining pressure. Figure 32 is based on a magnitude 7.5 earthquake and takes into account the percentage of fine material passing the number 200 sieve. Figure 33 was generated to show the effects of other magnitudes. Both Figures 32 and 33 apply to level ground conditions free of initial static shear stress. A soil element in and embankment or beneath a structure has an initial static shear stress and has an initial shear stress ratio of alpha defined as the shear stress on the horizontal plane divided by the effective vertical confining pressure. A correction factor to be applied to the blowcount resistance to account for initial static shear has been developed by Hynes et. al. (1988) based on laboratory tests on Folsom gravels, Figure 34. It is assumed that the initial static shear increases resistance to liquefaction. Finn (1991) points out that "this increase applies to resistance to cyclic mobility rather than to liquefaction, that is to non-contractive materials." Dense cohesionless soils would be expected to have greater resistance to deformation and retain their strengths when initial static shears are present. However Vaid and Chern, Reference 19, show for contractive materials the initial static shear decreases the resistance to cyclic loading. Loose soils would tend to experience an increase in contractiveness and a strength drop after reaching peak strength. The effect of a soils looseness in terms of relative density is shown in Figure 35. When a structure rests on loose sand factors of much less than 1.0 must be applied to the shear stress ratio. This in effect reduces the strength of the soil which can be counted upon for very loose deposits. This is of critical significance to Navy waterfront facilities. Increasing vertical confining pressures also has the effect of reducing the resistance to cyclic loading for the very loose sands, Reference 20. Figure 32 was developed for clean sands; to apply Figure 32 to silty sands the following corrections were proposed by Seed et. al., Reference 18, to be added to the penetration resistance:

| Fine Content | Value to add (blows/ft) |
|---|---|
| 10% | 1 |
| 25% | 2 |
| 50% | 4 |
| 75% | 5 |

In addition to the Standard penetration Test, the Cone Penetration Test has been related to liquefaction, Reference 22.

Of significant interest is that given the occurrence of low blowcounts what will be the resulting deformation state in a given earthquake? This is best explained in terms of the residual steady state strength of the soil. Soils at a density looser than critical state are contractive, generate pore pressures and suffer near complete loss of residual strength, line 1, Figure 36. Cyclic mobility is a term applied to cohesionless soils that are at a density greater than that at the critical void ratio; such soils are expansive or less contractive than loose soils under cyclic loading and exhibit a stress-strain behavior with limited deformation, line 2 Figure 36. Under loading dense soils tend to expand approaching the critical void ratio and loose soils tend to contract approaching the critical void ratio. The undrained steady state strength is maintained over a large range of strain. In 1988 Seed et. al., Reference 18, developed an estimate of residual strength as a function corrected blowcounts. It must be emphasized that this was developed from case histories and only is intended to give approximate values.

## DETAILED ANALYSIS OF SITE

### Site Analysis

It was of interest to examine the Treasure Island site to explore reasons for the site amplification. The data reported by Hryciw et. al. (Reference 5) was used as the starting point for this investigation. Additional information on the site was obtained from Reference 23 and is shown in Figures 37 and 38. Figure 37 shows the blowcounts from a Standard Penetration Test which are noted to be less than 10 for a significant portion of the profile. Figure 38 shows the undrained shear strength.

### SHAKE Analysis

Using the same soil profile defined in Reference 5, the soil column was analyzed for the Loma Prieta earthquake using the SHAKE85 computer program. The Yerba Buena Island record was used as the rock input motion and average cyclic shear strain was taken as 0.65 times the maximum shear strain. Professor Rollins, Reference 5, used SHAKE90 a more recent version of the same program. SHAKE90 uses strain dependent properties based on the ratio of shear modulus as a function of strain to the maximum shear modulus which occurs at low strain; multiple functions can be used for representation of sands and clays. SHAKE85 uses a function which is defined by equation; the user is limited to only 1 function for clay. These differences caused slight differences in the computed results. Figure 39 shows the NCEL data which agrees very closely with that of Reference 5. This establishes a benchmark control point from which a detailed analysis can be undertaken.

One point of considerable interest is the strain dependent properties for the Bay Mud. Rollins in Reference 24 uses data by Lodde, Reference 25, to define the strain dependent shear modulus ratio. Figure 40 shows a plot of the Bay Mud curve compared with the more normal values based on data provided in Reference 26. It can be seen that the Bay Mud has a significantly stiffer modulus with strain. Figure 40 also contains data from Mexico City, Reference 27 which is very similar to the Bay Mud behavior. The Mexico City clays were noted to be rather stiff at low strain. Note that distant earthquakes are low strain events. The SHAKE analysis was repeated using the less stiff values normally associated with clays as a substitute for the Bay Muds. Figure 41 shows that amplification does not occur. Strains in the analysis using the Bay Mud properties are in the range of 0.03 to 0.08 percent in the Bay Mud layers; this results in an effective shear modulus of about 60 percent of maximum with damping in the range of 0.06 to 0.12 of critical. However when typical clay data is used the shear modulus drops to about 10 percent of maximum and damping increases to 0.08 to 0.15 of critical. This explains why the stiffer Bay Mud properties do not attenuate the motion as does typical clay. Reference 24 evaluates the significance of shear wave velocity over a wide range and concludes that this is not a factor of high sensitivity. They also conclude that:

"the older bay sediments contributed very little to the ground amplification, the Bay Muds contributed somewhat, but by far the greatest contribution came from the fill material. However, this observation should not be understood to mean that the fill sand is inherently more prone to amplification than the Bay Mud, but rather that the fill is under lower confining pressure and by its surcharging effect, provides the Bay Mud with higher shear stiffness."

This author disagrees at least in part with this conclusion. It would appear from looking at Figure 39 that the amplification is occurring in the upper sand layer; however as shown above it is the stiff Bay Mud characterization which was shown to cause amplification. It is true that the confinement of the Bay Mud by the upper sand layer contributes to the stiffness of the Bay Mud. However the Bay Mud as characterized by Lodde in Reference 25 shows it to be stiffer with straining than would be expected. This is thought to be very significant. The San Francisco site and the Mexico City site both have clays that are substantially stiffer than would be expected. Reference 30 shows that the Plasticity Index for Bay Muds is in the range of 20 to 40 between 38 and 75 feet. The Plasticity Index for Mexico City clays was 30. Reference 31 shows data documenting that the shear moduls is stiffer with shear strain as the Plasticity Index increases. This little publicized data indicates that the stiffness of clay under cyclic loading should be increased to account for the Plasticity Ratio. The Plasticity Index is based on the amount of water required to transform a remolded soil from semisolid to a liquid state. It is a function only of the size shape and mineralogy of the soil particles and the pore water.

The effect of loose sand deposits is also clearly of interest since it is believed that these can contribute to amplification. This will be explored in the next section.

**Effects of Loose Surface Deposits**

To study the problem of whether loose sand deposits contribute to amplification a generic soil profile was constructed composed of layers of sand. The upper 70 feet was assumed to be composed of sand. The Pasadena 1952 earthquake record was applied to the bedrock normalized to the 0.05g level. A series of studies were conducted during which the upper layers was assumed composed of sand corresponding to a specific Standard Penetration Test blowcount. The blowcount was converted to shear modulus and the associated property used in the analysis. Figure 42 is taken from DM7 and is based on work by Ohsaki and Iwasaki, Reference 28 from 1972. This data is old and limited and should be replaced. Reference 29 is a new comparison and presents Figure 43 which shows other relations. For the purpose of this analysis the most representative average estimate was thought to be the Ohta relationship for sands. Figure 44 shows the acceleration profiles for various blow counts of the upper 70 foot zone. It is clear that the looser the zone the lower the shear modulus and the higher the amplification. Figure 45 illustrates the sharp increase in amplification when the blow count falls below 20.

**Larger Earthquakes And Treasure Island**

A question occurs as to how Treasure Island will respond to a larger level of ground shaking. Will there be a corresponding amplification in ground motion? To answer this the analysis described above was repeated for an earthquake at twice the amplitude of the Loma Prieta event. Figure 46 shows the expected site acceleration with depth. Doubling the bedrock motion did not double the surface acceleration. Nonlinear behavior resulting in a softer shear modulus combined with increased damping reduced the amplification from about 3 to about 2. The effect of the stiff Bay Muds at low strain produces maximum amplification at the low levels of ground shaking associated with those low strains. Thus the phenomenon is primarily associated with distant earthquakes having low levels of ground motion which excite the soil deposits at low levels of strain to produce the amplification. This phenomenon can be predicted by cyclic shear tests to establish the shear modulus of the clays under various levels of loading.

Figure 47 gives guidance based on Reference 31 for estimating the effect of Plasticity Index on the shear stiffness of clays. This figure is suggested for use on Navy coastal sites where high plasticity soils like Bay Muds can be found.

## SETTLEMENTS OF LIQUEFIED SAND DEPOSITS

As noted above settlements occurred on Treasure Island and caused damage. Had the Loma Prieta earthquake lasted longer it would have induced additional cycles of loading in the soil which would have significantly increased the deformation. As a saturated cohesionless soil in loose condition undergoes cyclic shear pore water pressures begin to increase. As the pore pressures increase, the effective confinement is reduced, which in turn causes a reduction in shear modulus that makes it easier to deform the soil for a given level of loading. Unfortunately this mechanism feeds upon itself such that as the soil softens with reducing shear modulus, large deformations can occur with the onset of liquefaction. Soils which exhibit restrained deformations under one level of shaking can exhibit substantially greater deformation under an event slightly longer in duration or higher in amplitude. Navy bases situated on loose saturated soils are highly vulnerable to damage unless the structures have been designed to mitigate this effect. Mitigation whether in the form of ground improvement or use of piles is very expensive. While we can predict the occurrence of liquefaction on level ground away from the structure with a high enough degree of certainty, prediction of liquefaction beneath and around the structure is substantially more complicated and not adequately defined. Prediction of the deformation state (settlements and ground flow) is even more undefined. This is a major area of concern for the Navy since we have numerous existing buildings built before liquefaction was recognized as a problem. New construction is often forced to include the most conservative foundation from a lack of understanding of the liquefaction deformation.

Liquefaction deformation includes two basic components: settlements resulting from compaction of loose saturated deposits with an outflow of pore water under shear induced pressures, and lateral spreading of the ground from flow of reduced strength liquefied deposits associated with minor slopes. The estimation of deformation, while a topic for research by many, is lacking reliable accurate procedures.

**Settlement Estimation**

The state of the art for settlement prediction of liquefied deposits consists basically of empirical data relating shear strain with volumetric strain. In Reference 32, Tokimatsu and Seed present data shown in Figure 48 which relates the volume strain induced by a magnitude 7.5 earthquake as a function of effective cyclic shear strain. They suggest that other magnitudes may be scaled by use of data in Figure 49. The authors apply this data to an estimation of settlement shown in Figure 50. They divide the soil deposit into layers, they estimate the shear modulus based on the ratio of shear at a strain level to maximum shear strain, and then use relative density or blowcount information to compute volumetric strain using Figure 48. The volumetric strain is adjusted to the earthquake magnitude based on Figure 49 and then corrected for multi-directional effects. Settlements are estimated from the volumetric strain of each layer and totaled.

The basic procedure outlined by Tokimatsu and Seed can be substantially simplified if a site analysis using the program SHAKE is used. Shake computes the effective shear strain as illustrated in Figure 51 for the Treasure Island site. The calculations can be improved by automation in the form of a spread sheet. A program was prepared , LIQSS which performs the settlement calculation using data from Reference 32. Figure 52 shows the spread sheet for the example shown in Figure 50. The computed settlement of 4.24 inches for a magnitude 7.5 event is reduced by 0.8 for a magnitude 6.6 event and is 3.22. The value of 3.22 is slightly less than the 3.37 inches computed by Reference 32. The difference is that LIQSS uses a finer interpolation algorith for magnitude correction.

Data from the SHAKE analysis of Treasure Island was used with blowcount data of the site, Figure 37 to predict the settlements at location UM10 on Treasure Island. The results are shown in Figure 53 using the low estimates of blowcount, Figure 54 using average estimates of the blowcount and Figure 55 using high estimates of the blow count. Magnitude corrected settlements range from 4.8 inches to 12.7 inches with a value of 6 inches for average values. This value is about the same as the 6 inches estimated from Figure 37.

A procedure has been developed using the data from Reference 32 using the Tokimatsu and Seed concept but modified to use the output from SHAKE . The procedure gave the correct order of magnitude of settlements for Treasure Island when average blowcounts were used. However the blowcount data is imprecise and the range of possible values using upper and lower estimates is large. For this reason NCEL is sponsoring a research study at the Waterways Experiment Station to bring together a group of experts to evaluate the available data and attempt to refine the process. The LIQSS computer program is an interim solution to estimating settlements until better procedures can be developed.

## CONCLUSION

This report has presented a detailed study of the response of the Treasure Island site during the Loma Prieta earthquake. It shows that the behavior during the earthquake was as might be expected given the loose soil conditions. An extensive discussion of settlements and liquefaction was presented. There was an extensive discussion of site improvements techniques used at Treasure Island and the resulting damage. The site was analyzed using the computer program SHAKE which was shown to give an accurate assessment of site response and soil amplification. Choice of appropriate material properties is essential for an accurate assessment, although the analysis was shown to be rather insensitive to shear velocities. SHAKE should be used as a preliminary screening tool since it does not consider material softening with development of pore water pressures; nor, does it include any 2-dimensional effects such as sloping bedrock conditions. Caution should be used in using code type spectra for soft site conditions. The effect of initial static shear stress from foundations was investigated where it was shown that for loose soils the initial shear reduces strength rather than increases it as might be expected for dense sands.

Using the data developed by Tokimatsu and Seed an automated procedure was developed for estimating liquefaction induced settlements. The procedure was applied to the Treasure Island site using the Loma Prieta earthquake and results agree with observed settlements. However the large spread in blowcount data give a large range in estimated settlements. This problem is under study.

Significant findings are:

- Buildings in areas which settled less than 6 inches generally experienced only minor damage.

- For loose sands the initial static shear caused by foundations results in a reduction of strength.

- Site improvement at least during moderate events was very effective. Alternative procedures appear to have performed equally well.

- SHAKE can predict site response and soft site amplification given an accurate assessment of the subsurface soil properties.

- Treasure Island amplification was shown to be dependent upon the Bay Mud properties.

- San Francisco Bay Muds resembles the Mexico City clays in being stiffer with straining than otherwise expected. This is related to its Plasticity Index

- Loose sands cause site amplification especially when blow counts of less than 20 occur in the upper layers.

- The LIQSS program using data from Tokimatsu and Seed gives reasonable estimates of settlement when average blowcount data is used.

**RECOMMENDATIONS**

The following are recommended:

- That Navy review soil borings for bases in seismically active regions to identify regions where clay deposits are prevalent.

- That the shear modulus properties of the clay under various levels of cyclic loading be obtained from soils reports if available. If not available tests should be conducted to determine whether those clays exhibit high stiffness as found in San Francisco and Mexico City. Screening should include evaluation of Plasticity Index

- That Figure 47 be used for high plasticity clays at Naval waterfront sites.

- That Navy support deep instrumentation arrays. Such an array will yield information on actual bedrock motion and amplification. NCEL and Western Division have jointly supported and assisted researchers from Brigham Young University and the University of New Hampshire perform borings and field tests at Treasure Island. NSF is providing funding for installation of a deep instrument array on Treasure Island.

- That Navy pursue NSF funding for an array in the San Diego area.

- LIQSS should be used as an interim approximate means for estimating settlements. Upper and lower bound blowcount should be used to give an estimation of the range of possible settlements.

## ACKNOWLEDGMENTS

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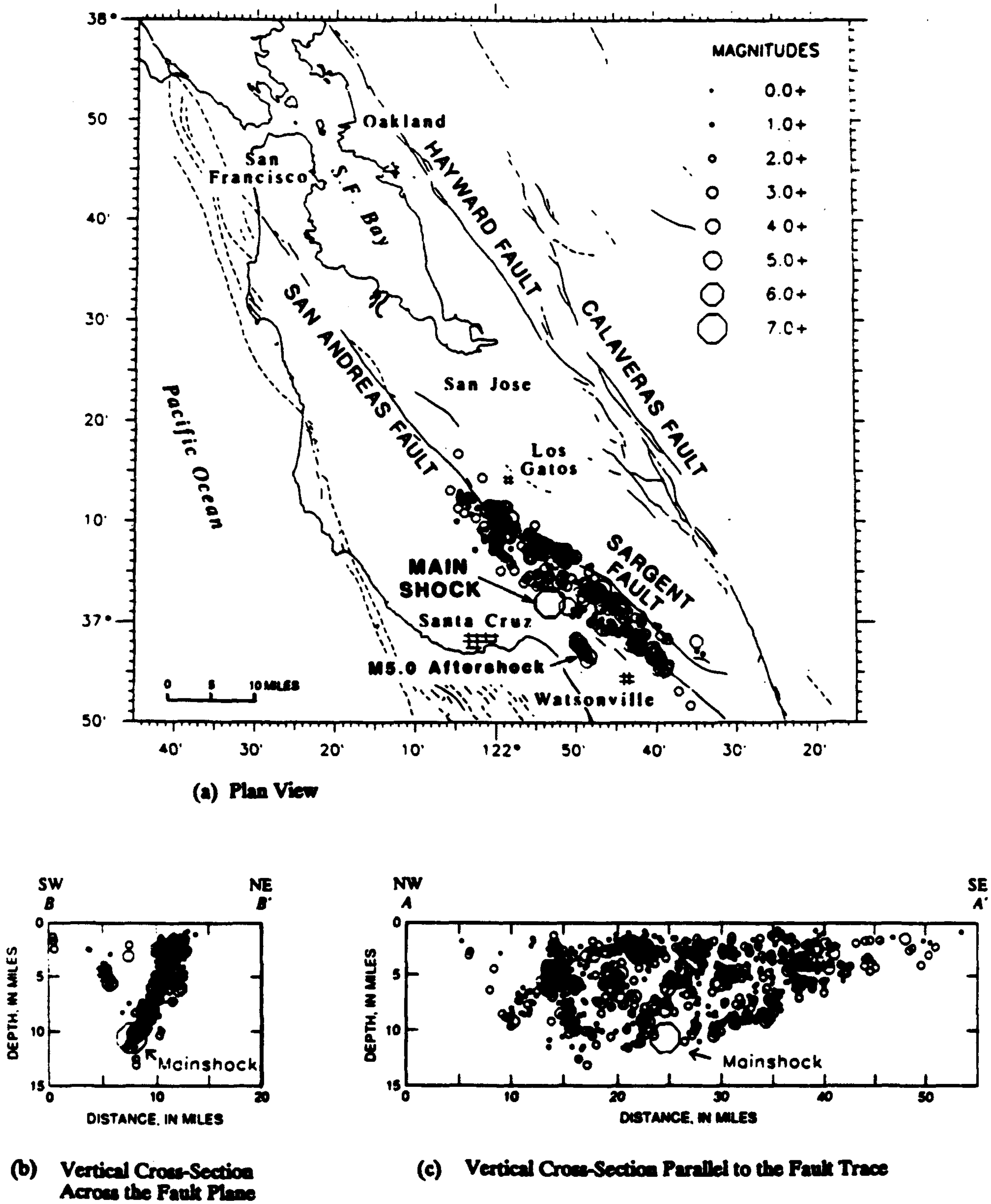



Figure 1. Location of Loma Prieata earthquake and aftershocks. (Reference 2)

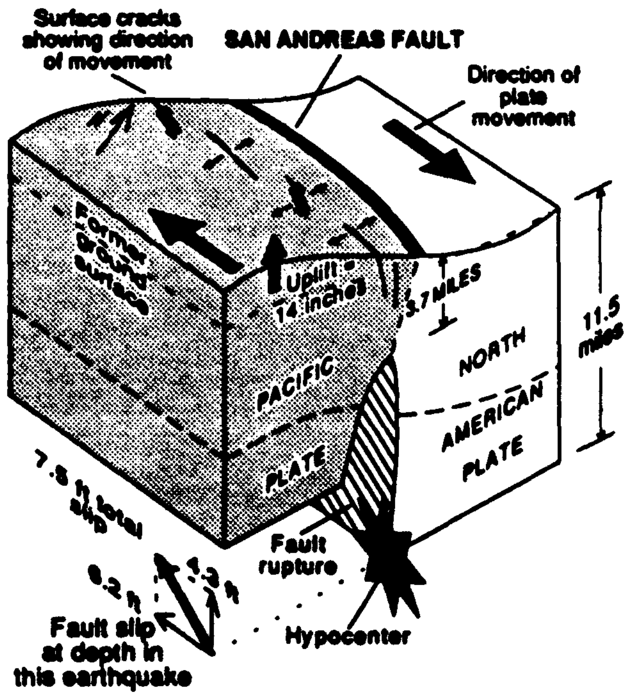



Figure 2. Illustration of fault movement (Reference 2)

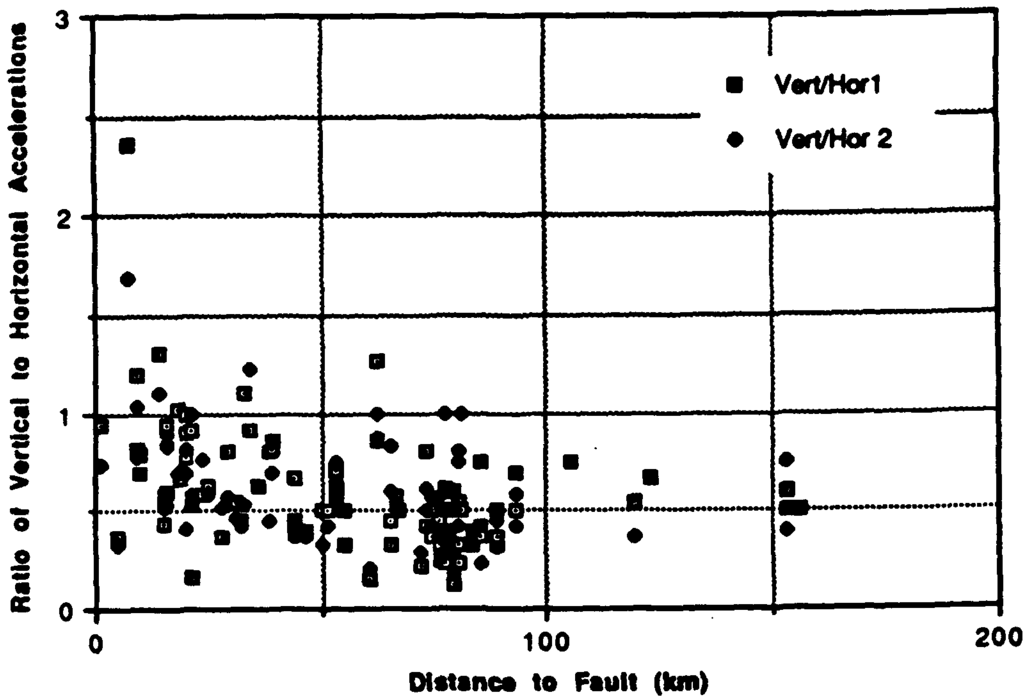



Figure 3. Ratio of vertical to horizontal acceleration, both components considered. (Reference 1)

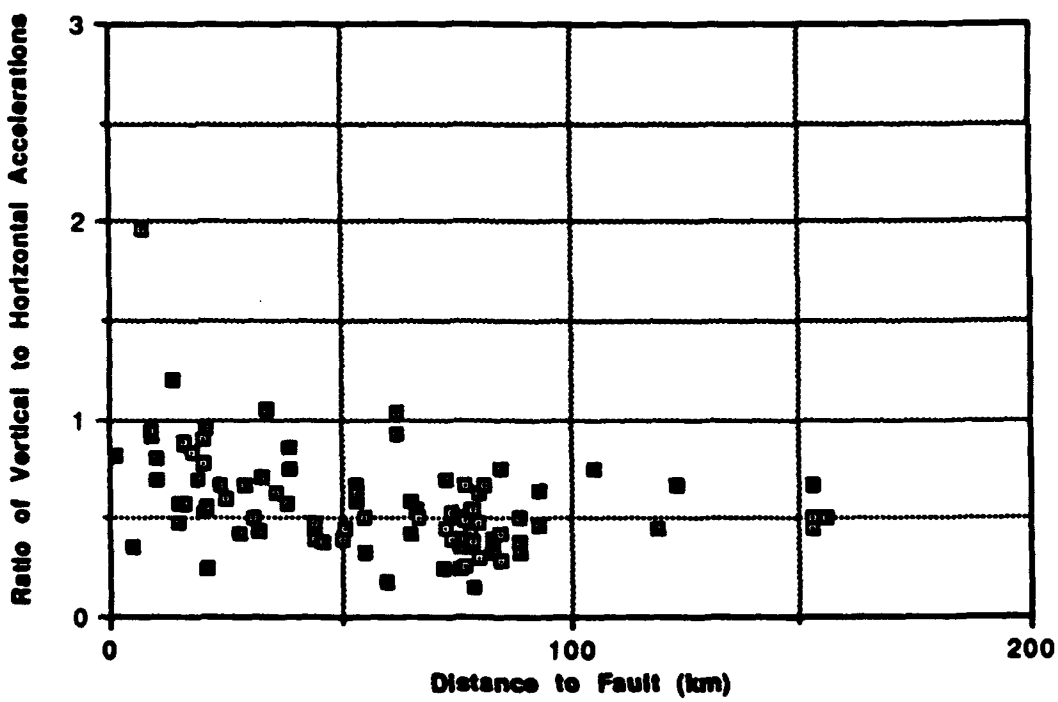



Figure 4. Ratio of vertical to horizontal acceleration, average of horizontal components. ( Reference 1)

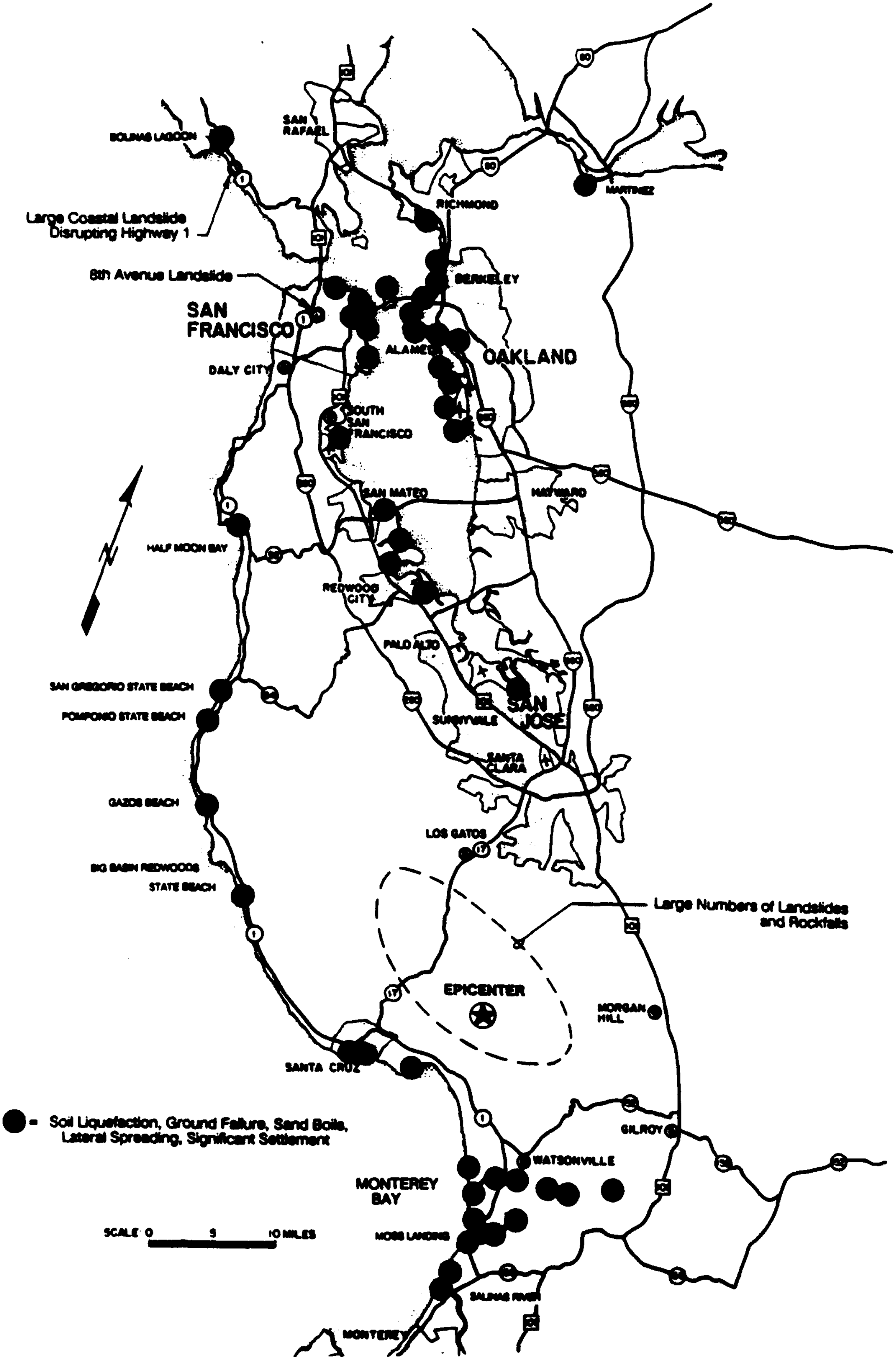



Figure 5. Map of affected area. (Reference 2)

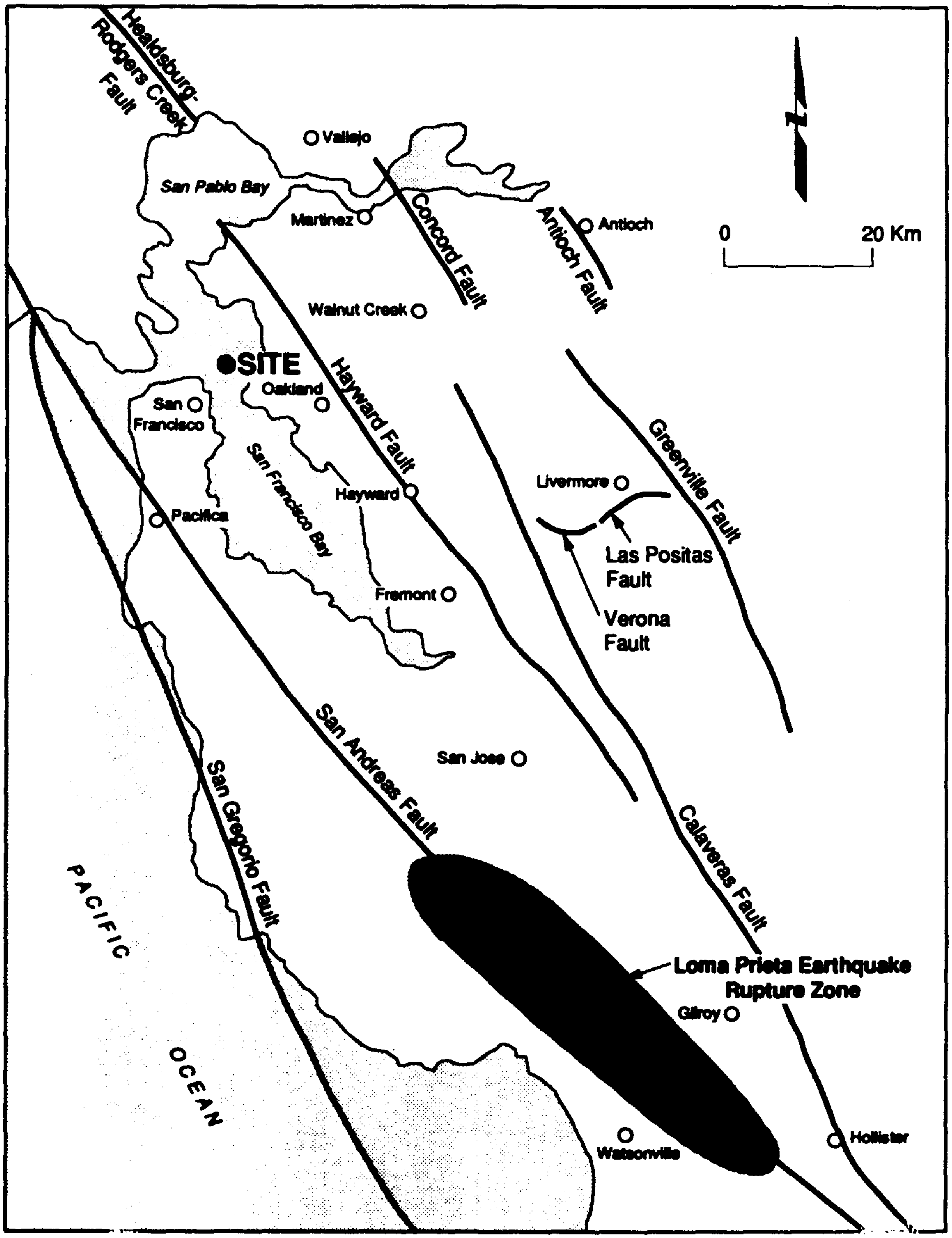



**Figure 6. Map of active faults in San Francisco Bay area**

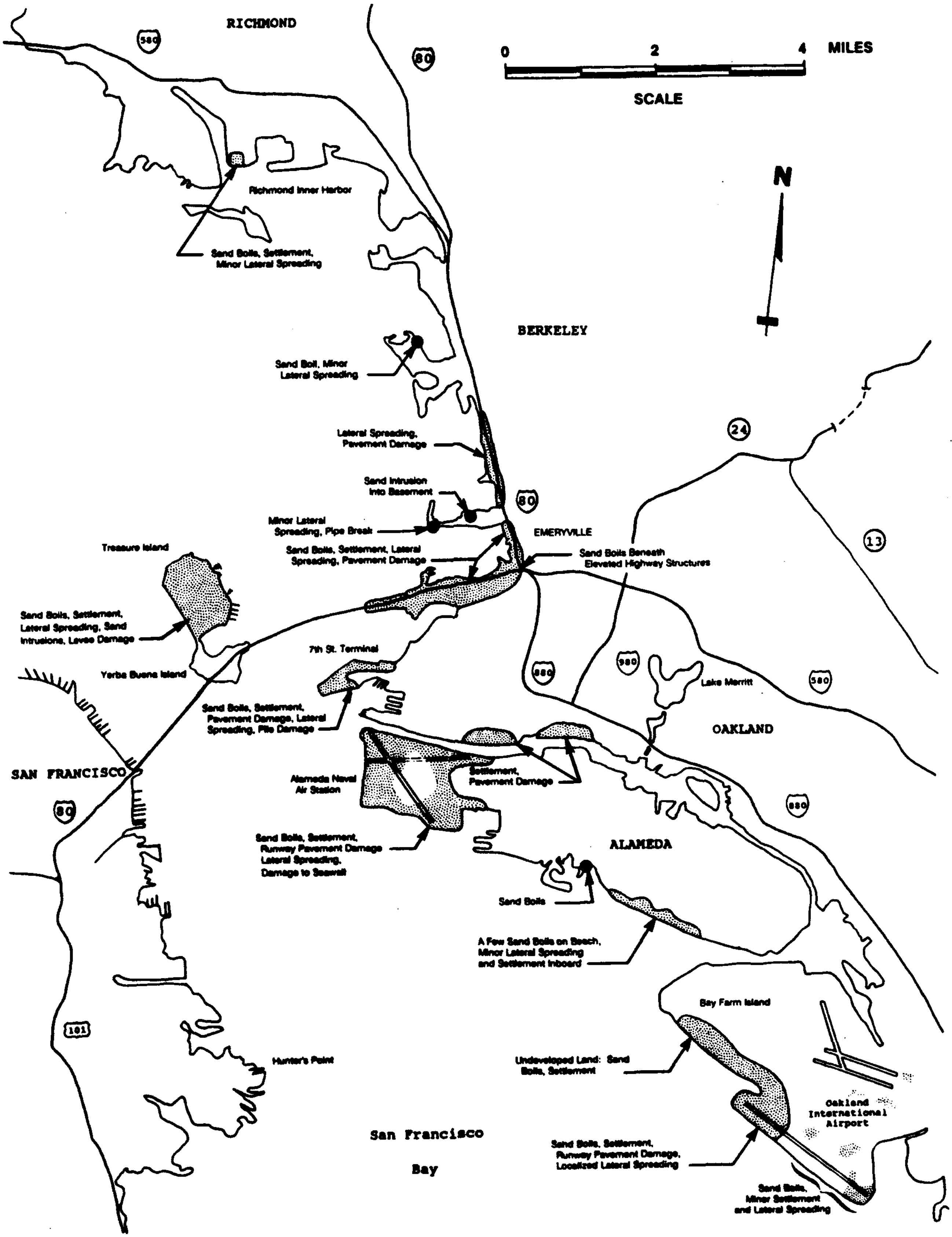



**Figure 7. SanFrancisco Bay showing Treasure Island and soil liquefaction. (Reference 4.)**

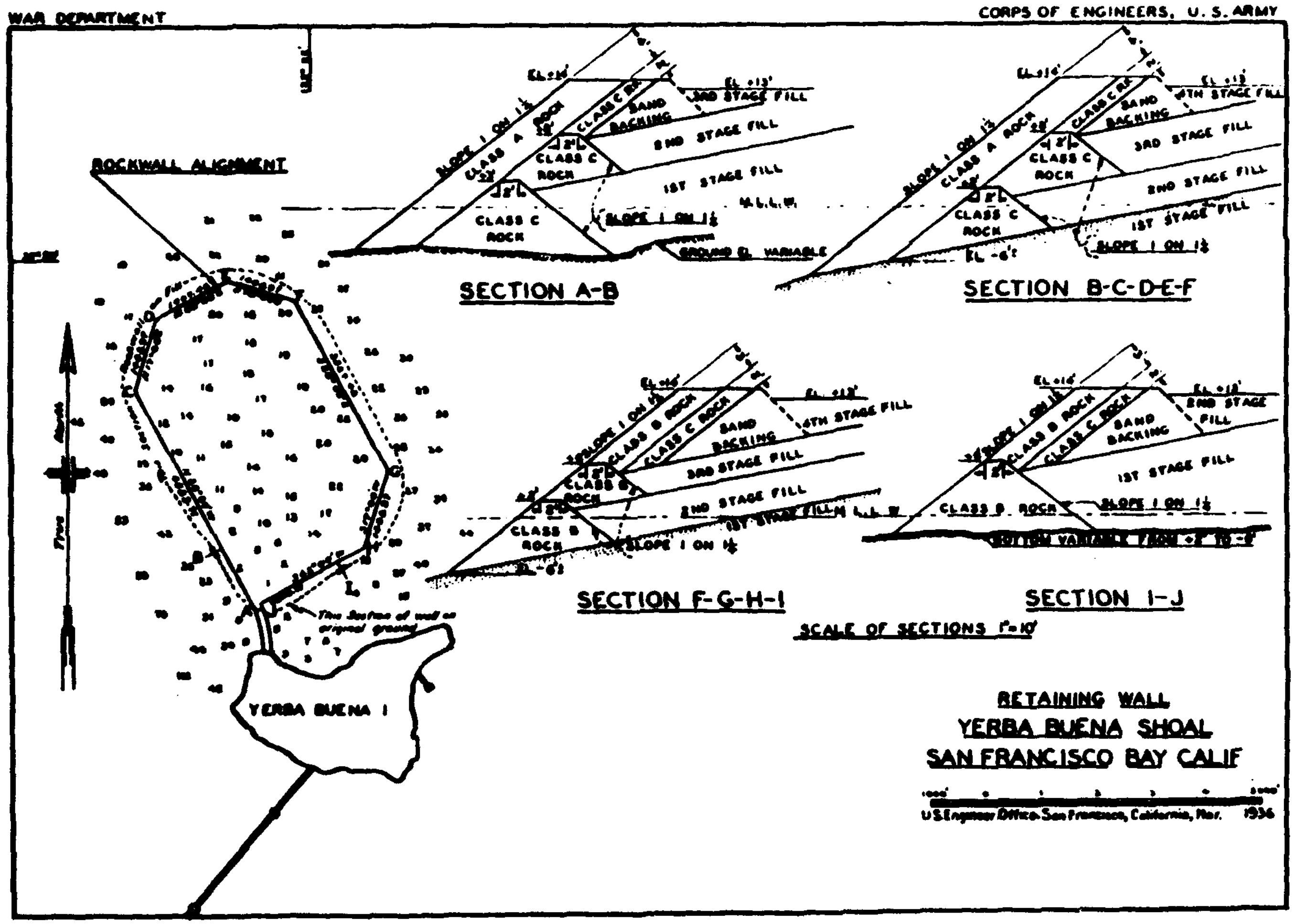



Figure 8. Perimeter dike. (Reference 7)

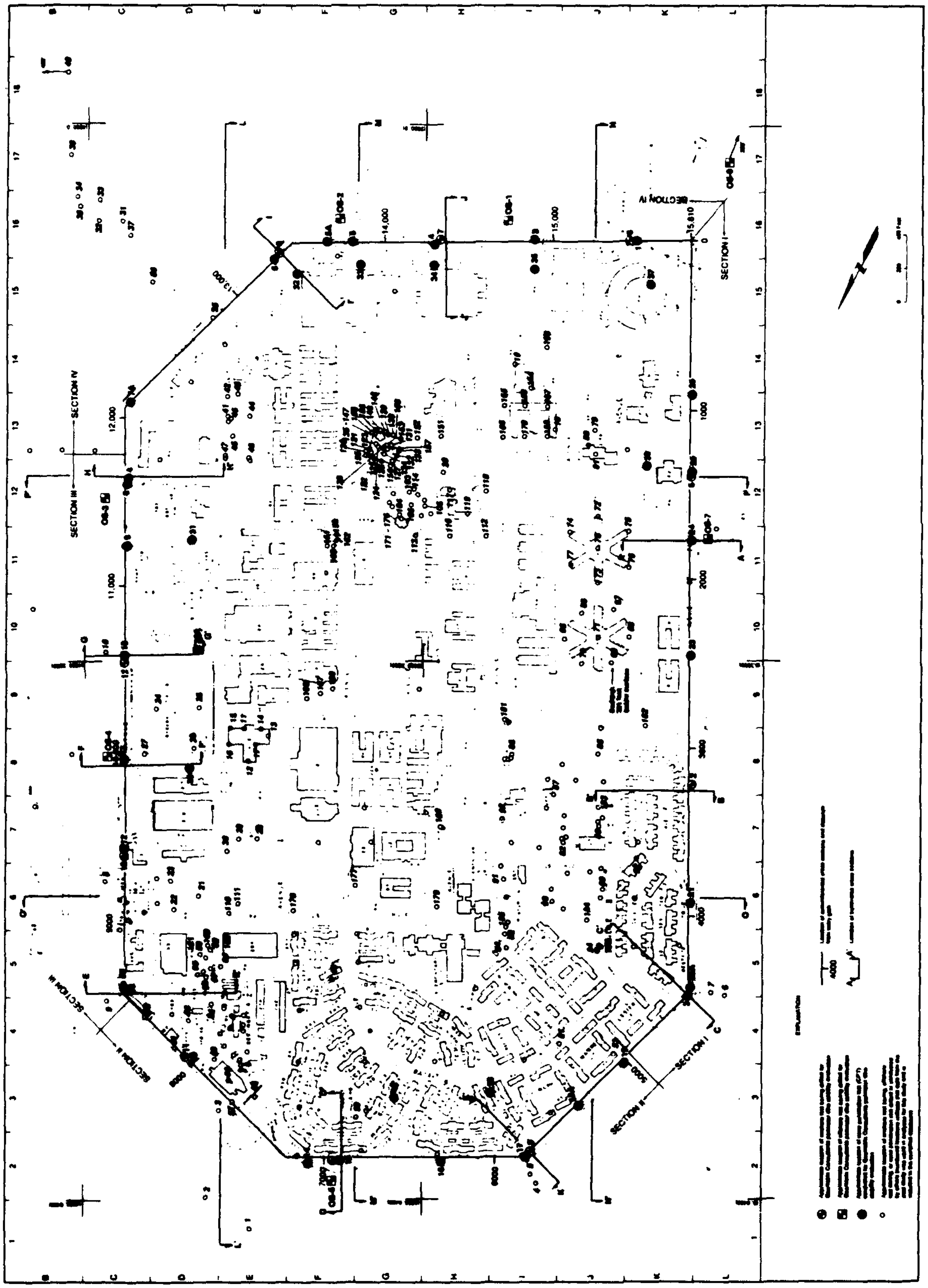


Figure 9. Treasure Island Site plan. (Reference 4)

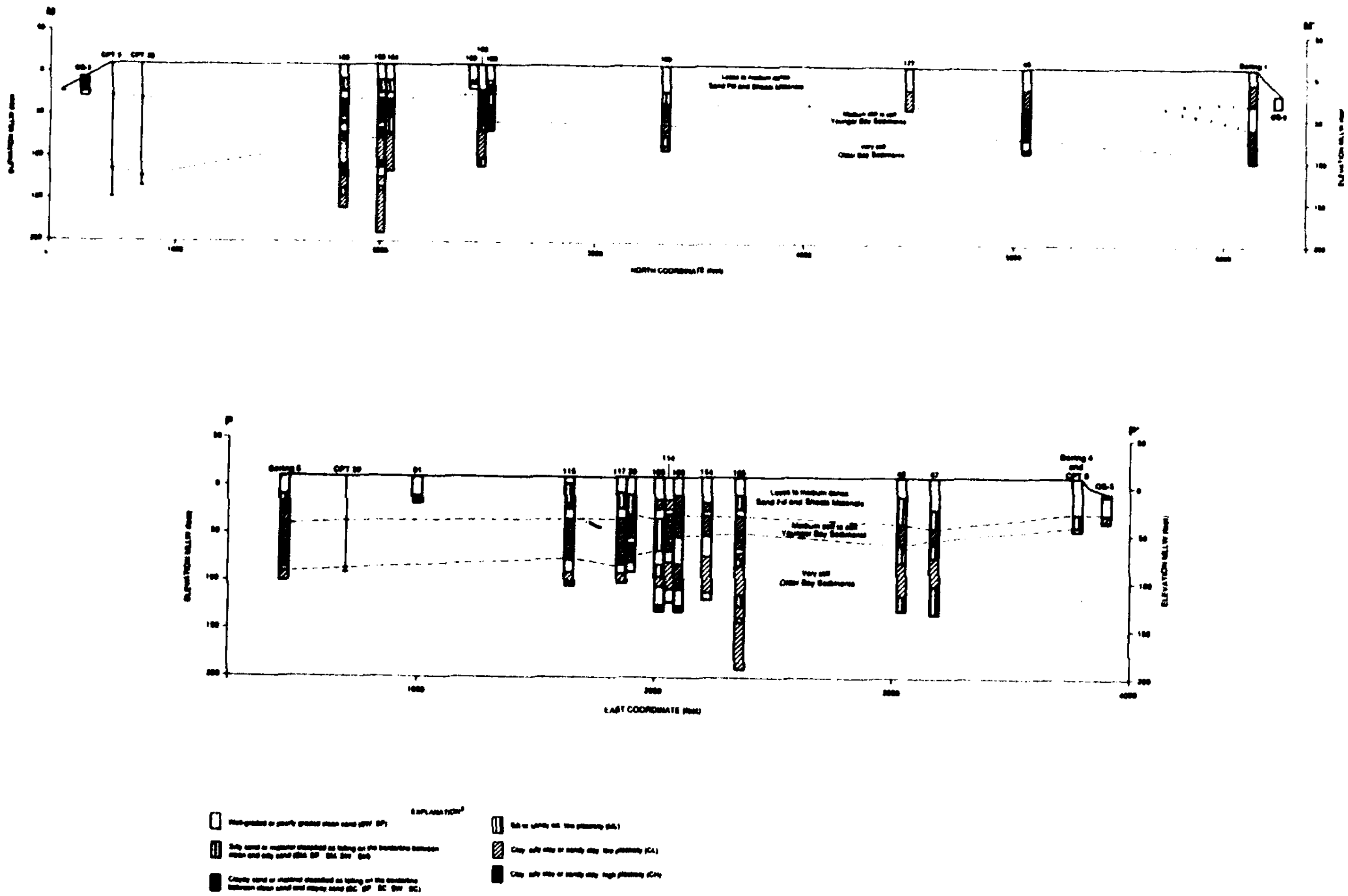



Figure 10. Stratigraphic cross section. (Reference 4)

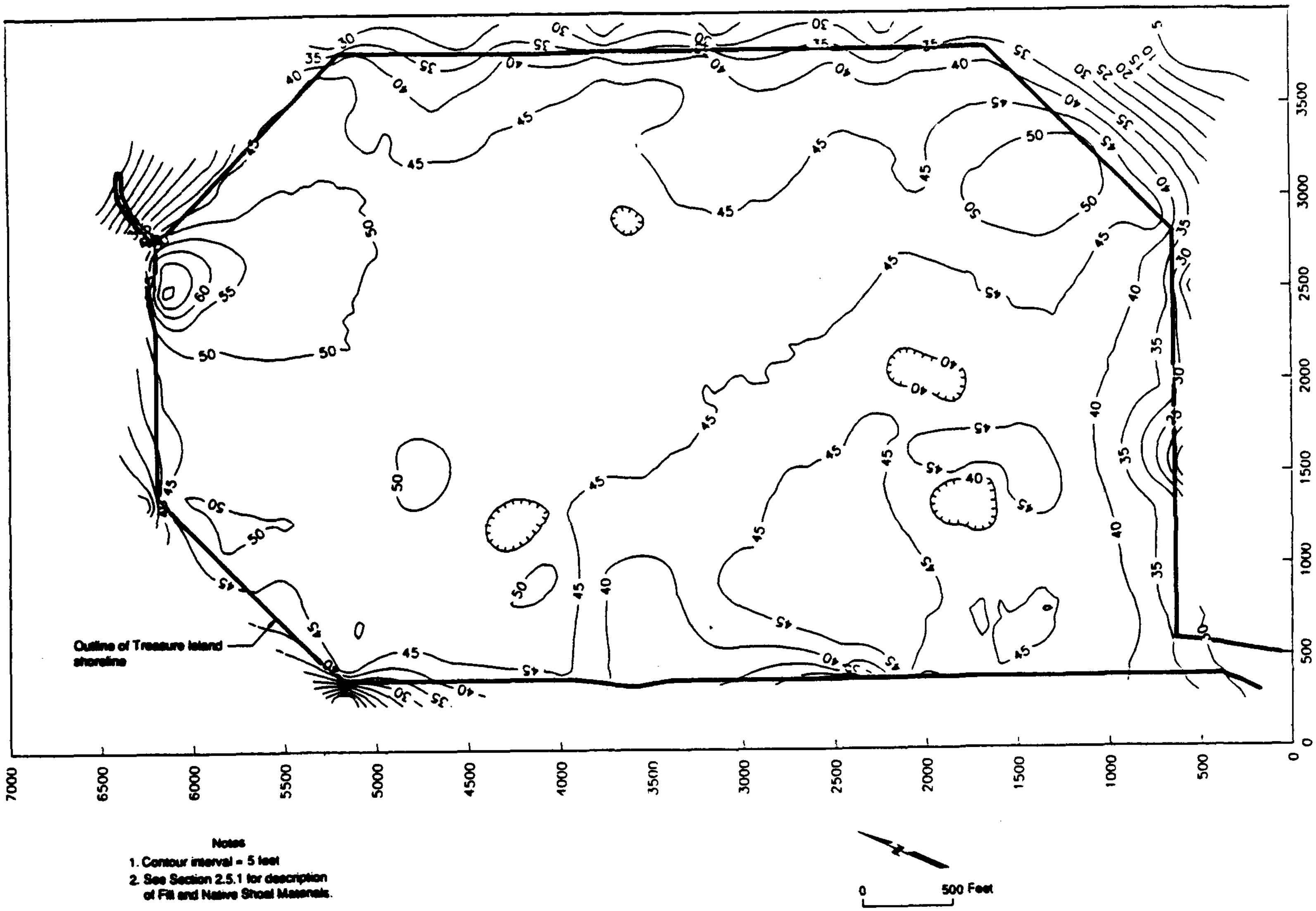



Figure 11. Thickness of fill and native shoals materials. (Reference 4)

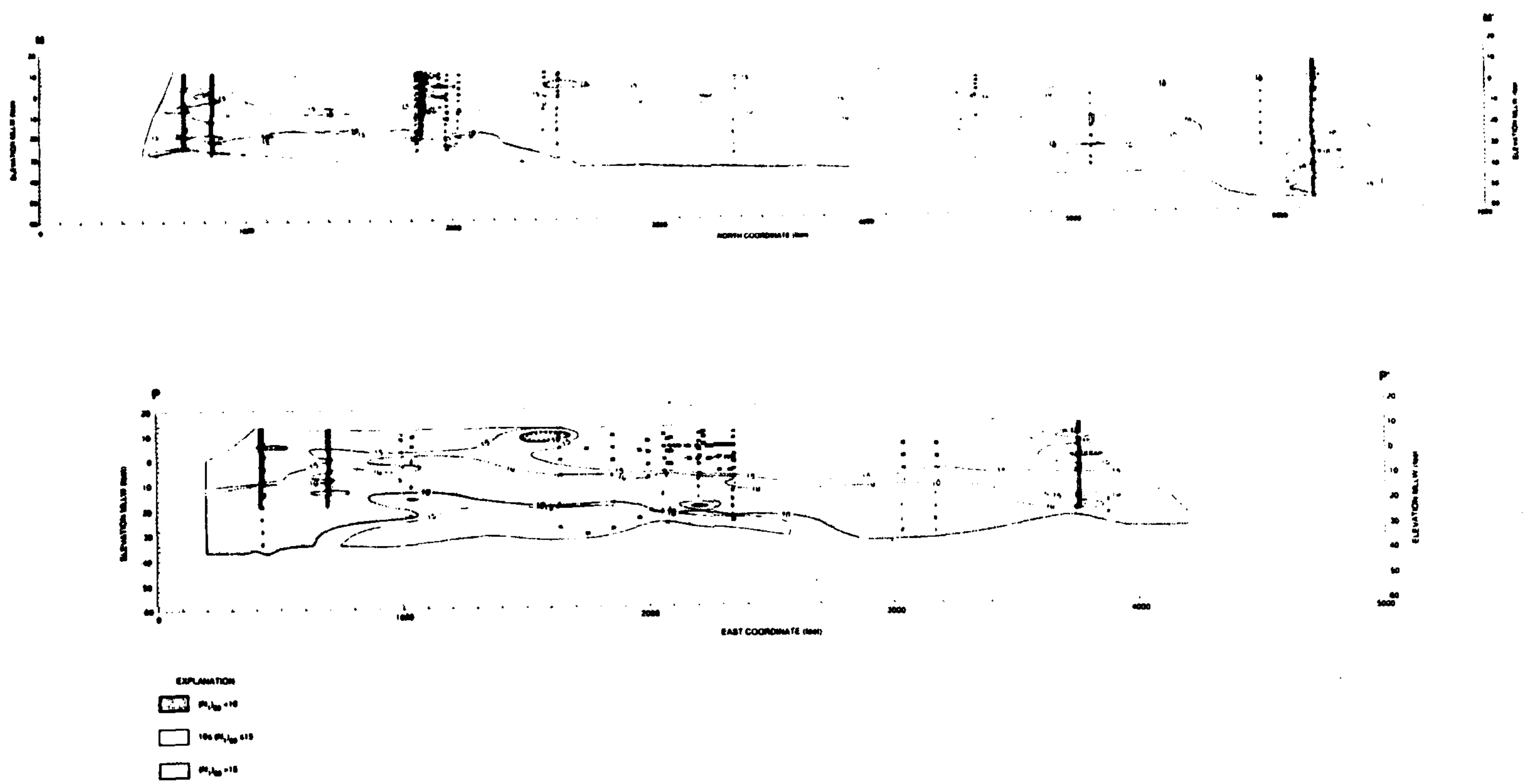



Figure 12. Standard Penetration Test corrected blowcounts in sand fill and native shoals materials. (Reference 4)

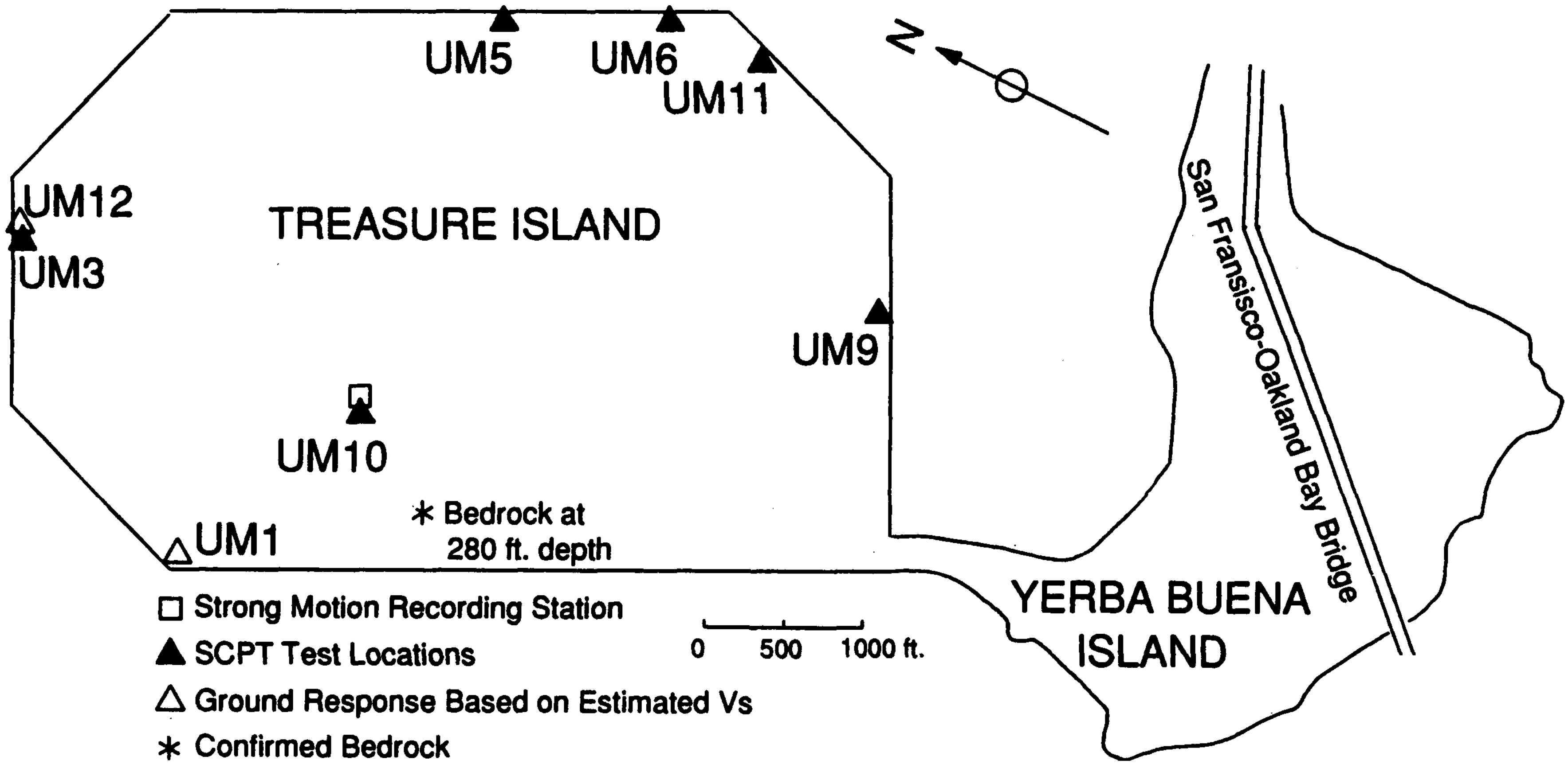



Figure 13. Treasure Island and Yerba Buena showing boring locations. (Reference 5)

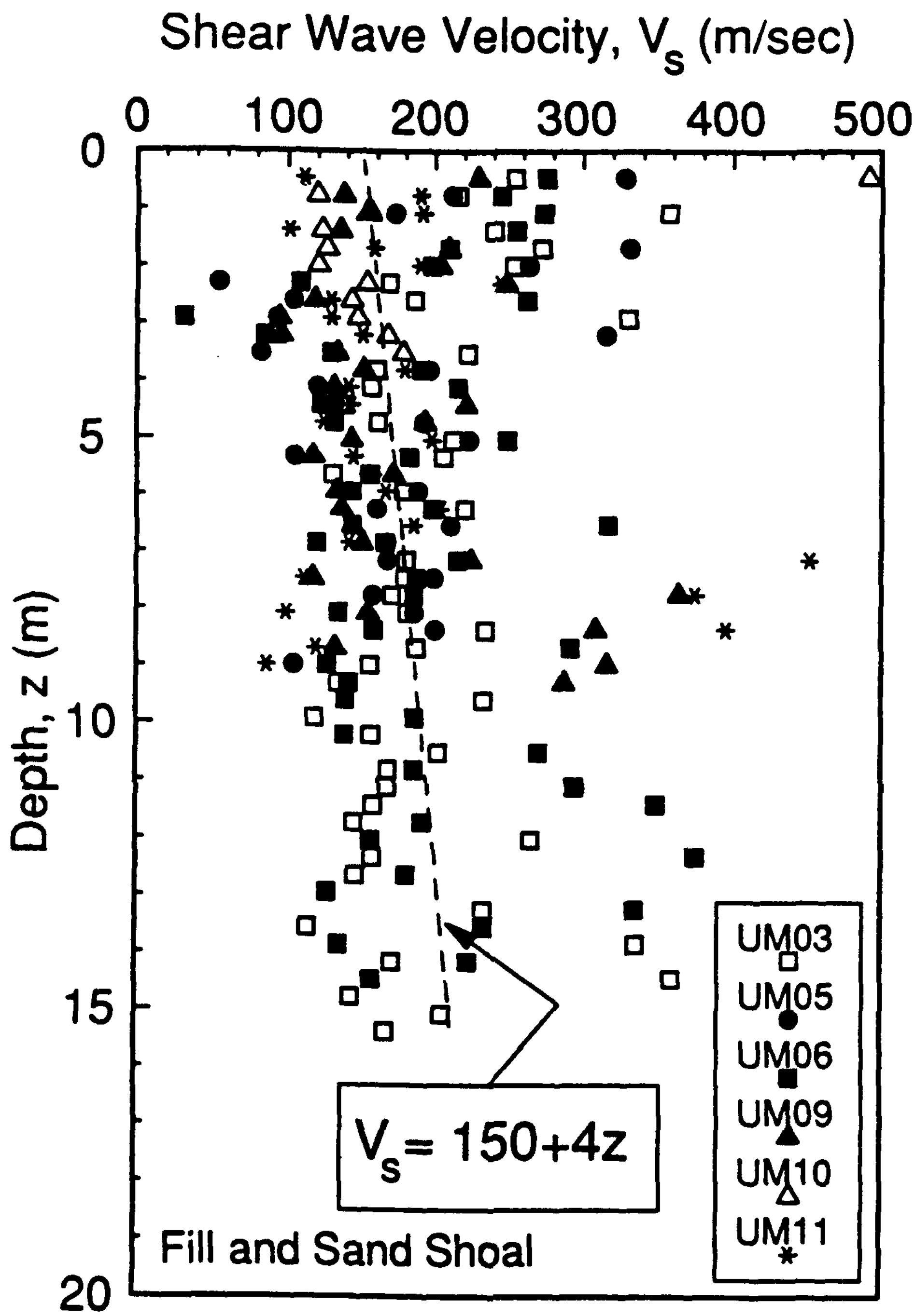



Figure 14. Shear wave velocity in fill and shoal sands. (Reference 5)

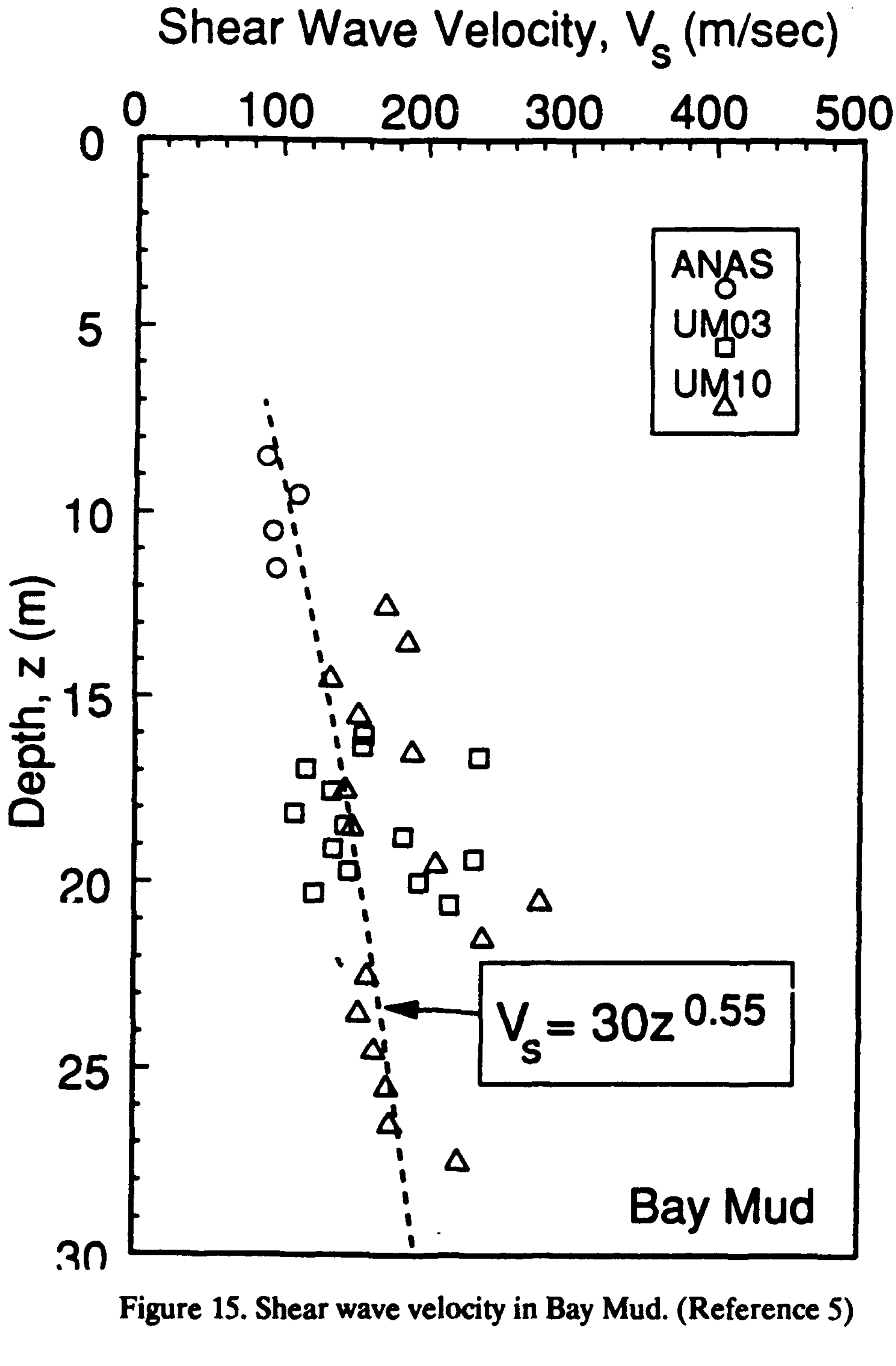



Figure 15. Shear wave velocity in Bay Mud. (Reference 5)

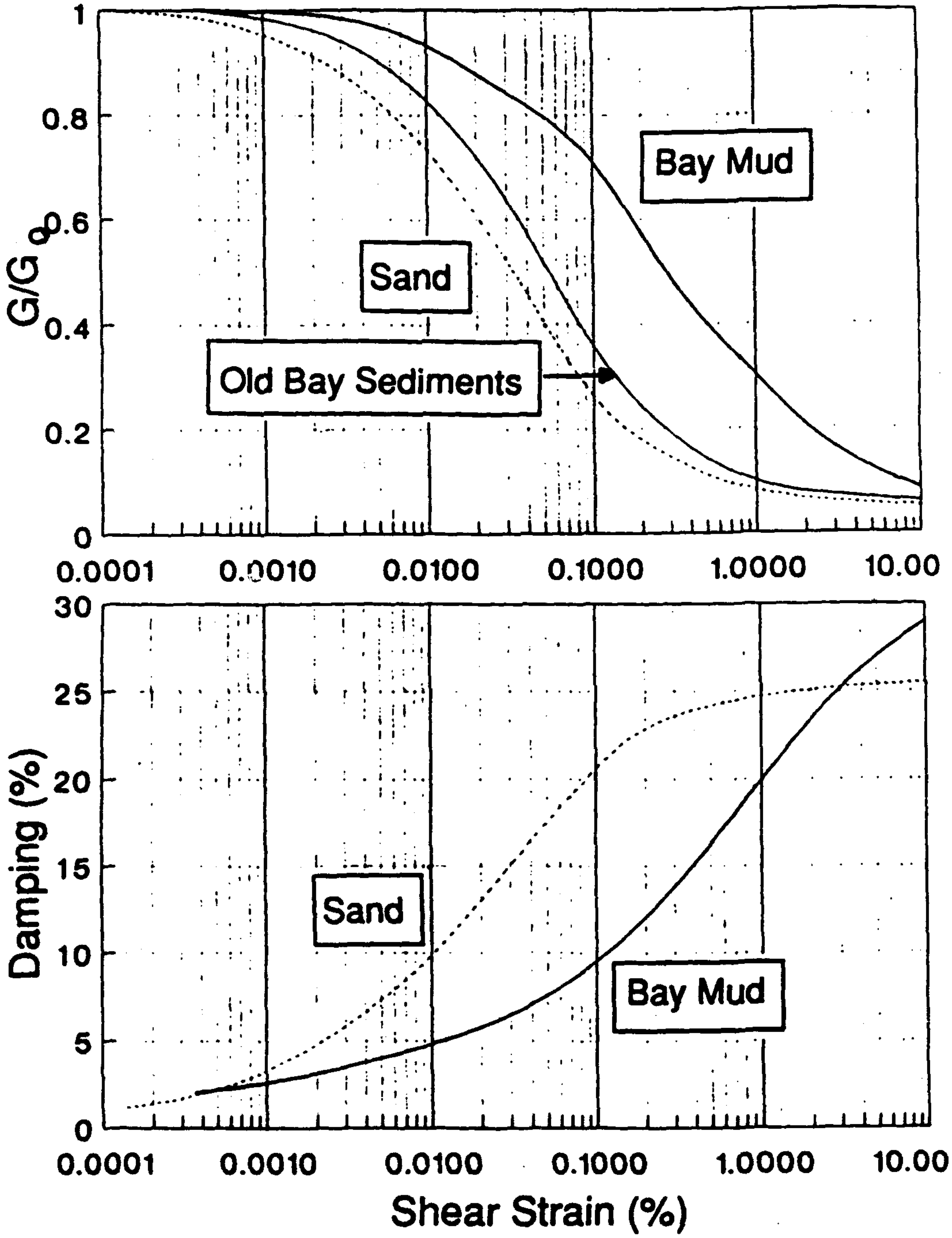



Figure 16. Normalized shear modules and damping with strain. (Reference 5)

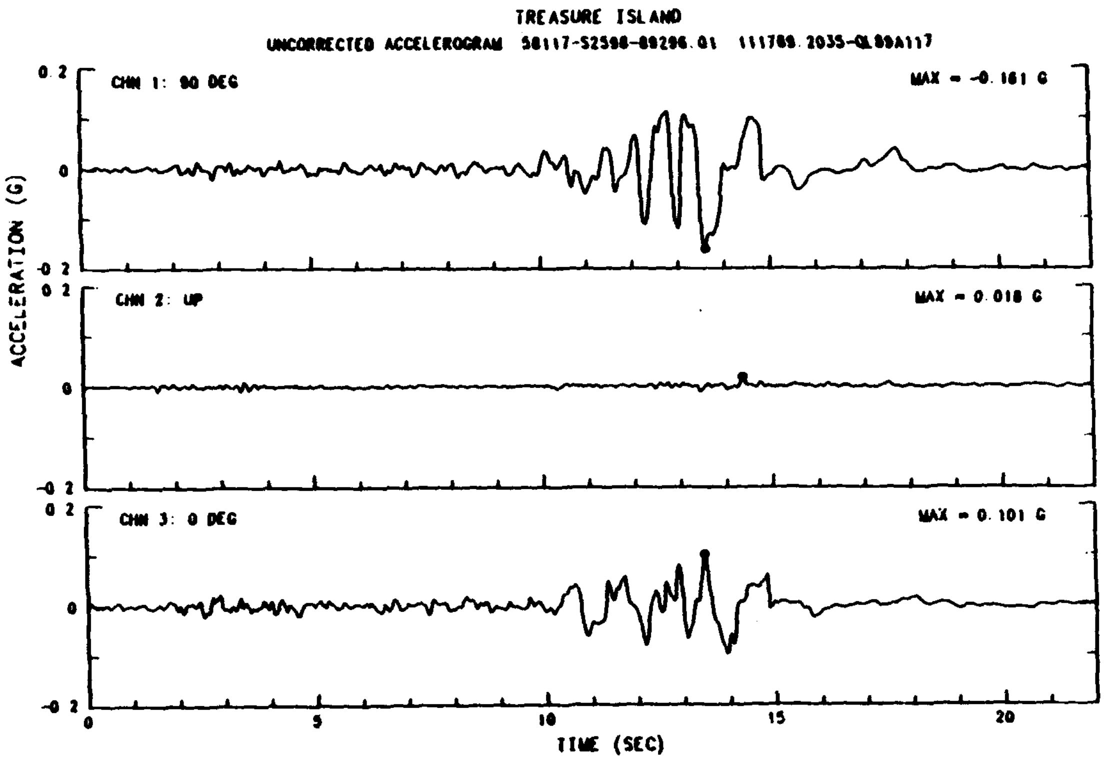



Figure 17 Treasure Island uncorrected accelerogram. (Reference 1)

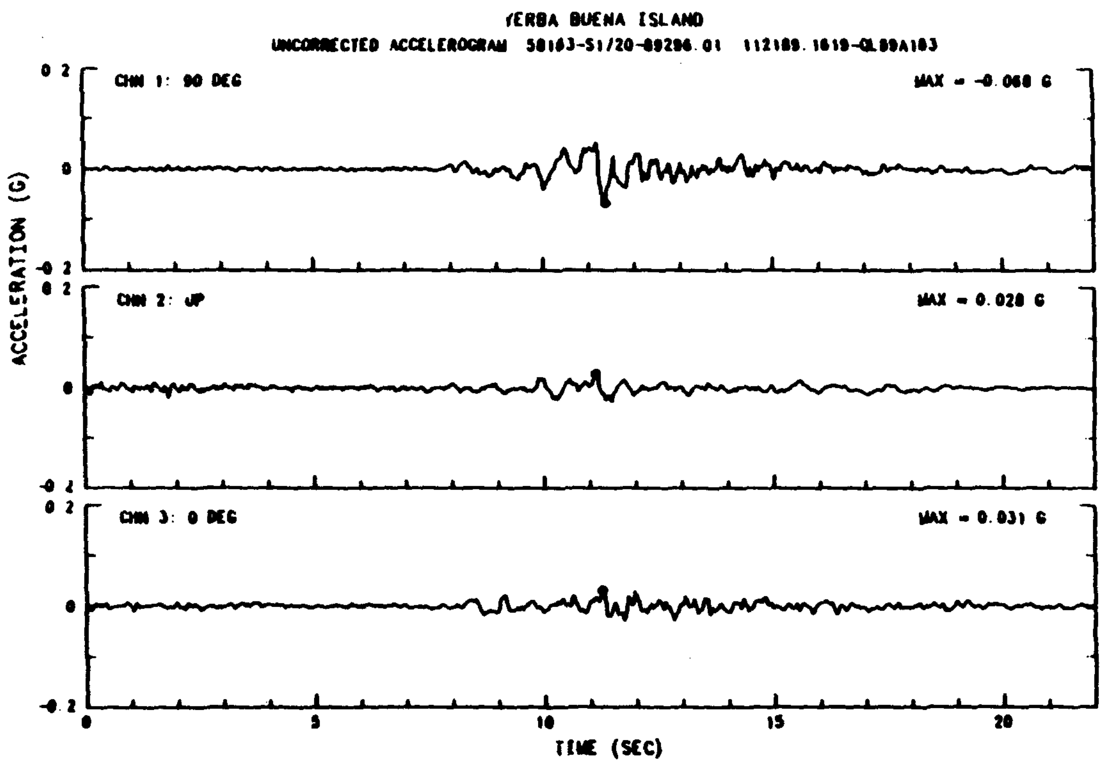



Figure 18. Yerba Buena uncorrected accelerogram. (Reference 1)

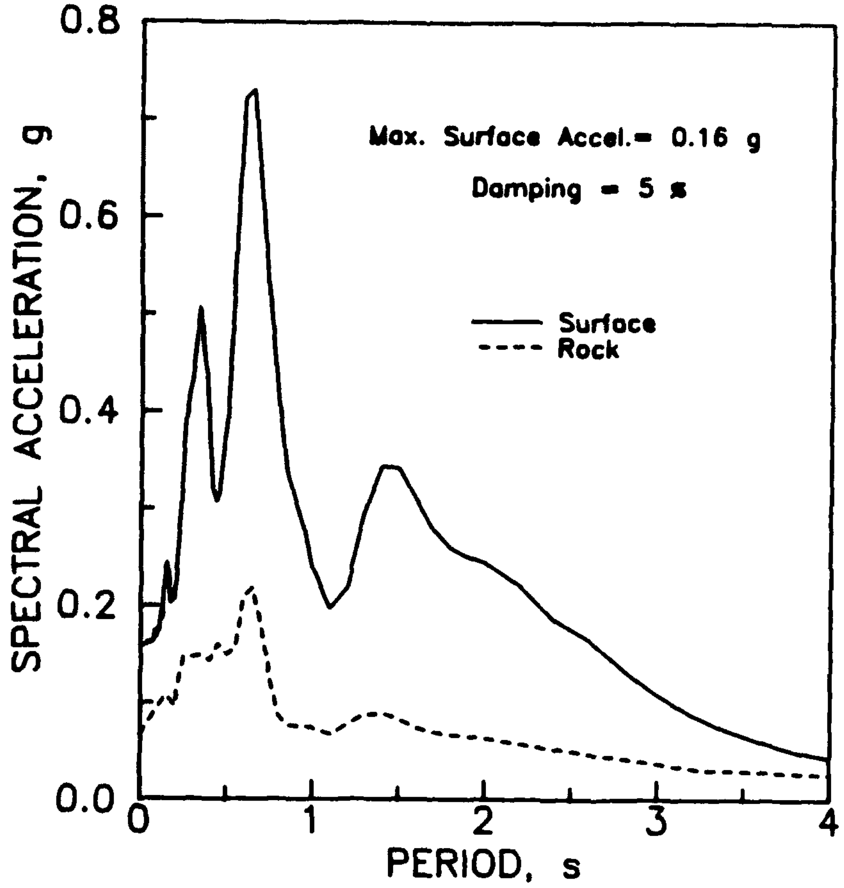



Figure 19. Response spectra, Treasure Island and Yerba Buena, 0 degrees(Reference 2)

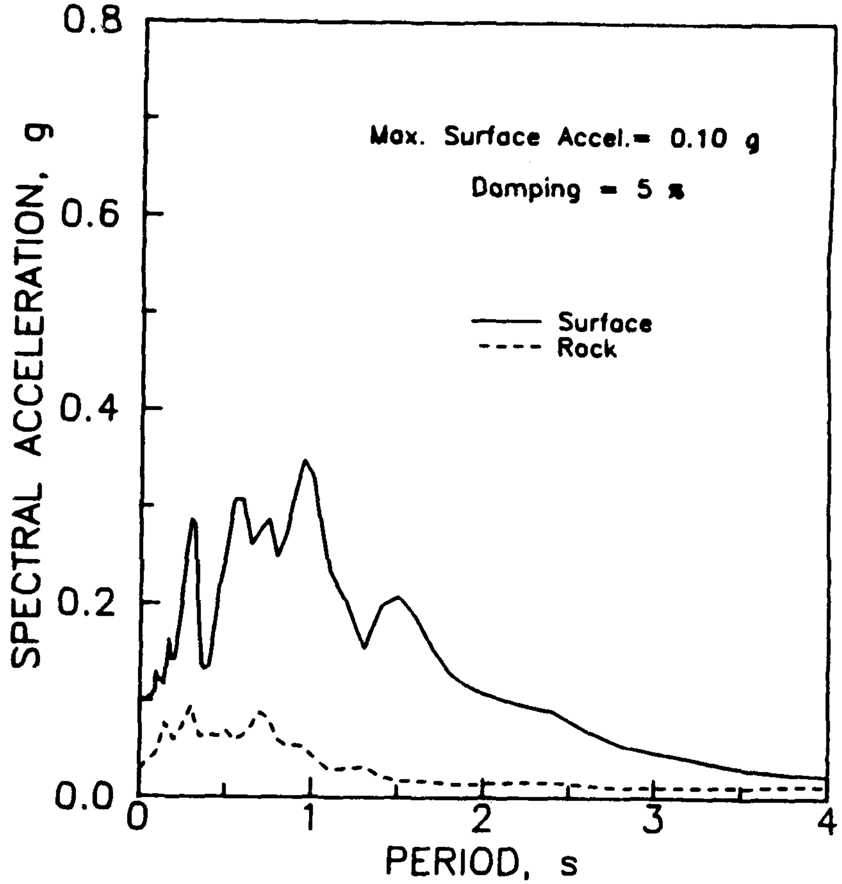



Figure 20. Response spectra, Treasure Island and Yerba Buena, 90 degrees. (Reference 2)

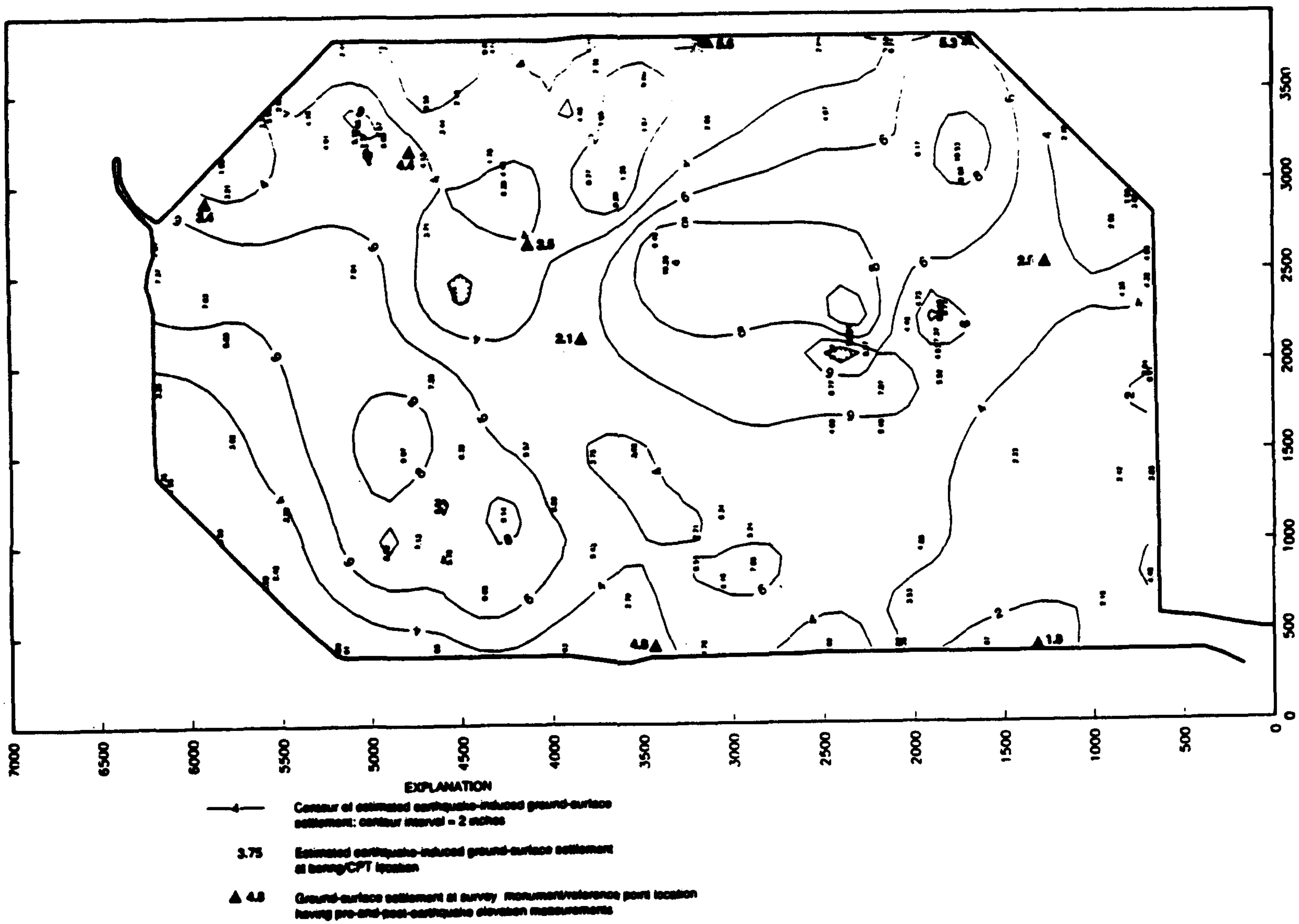



Figure 21. Shaking induced compaction settlement. (Reference 4)

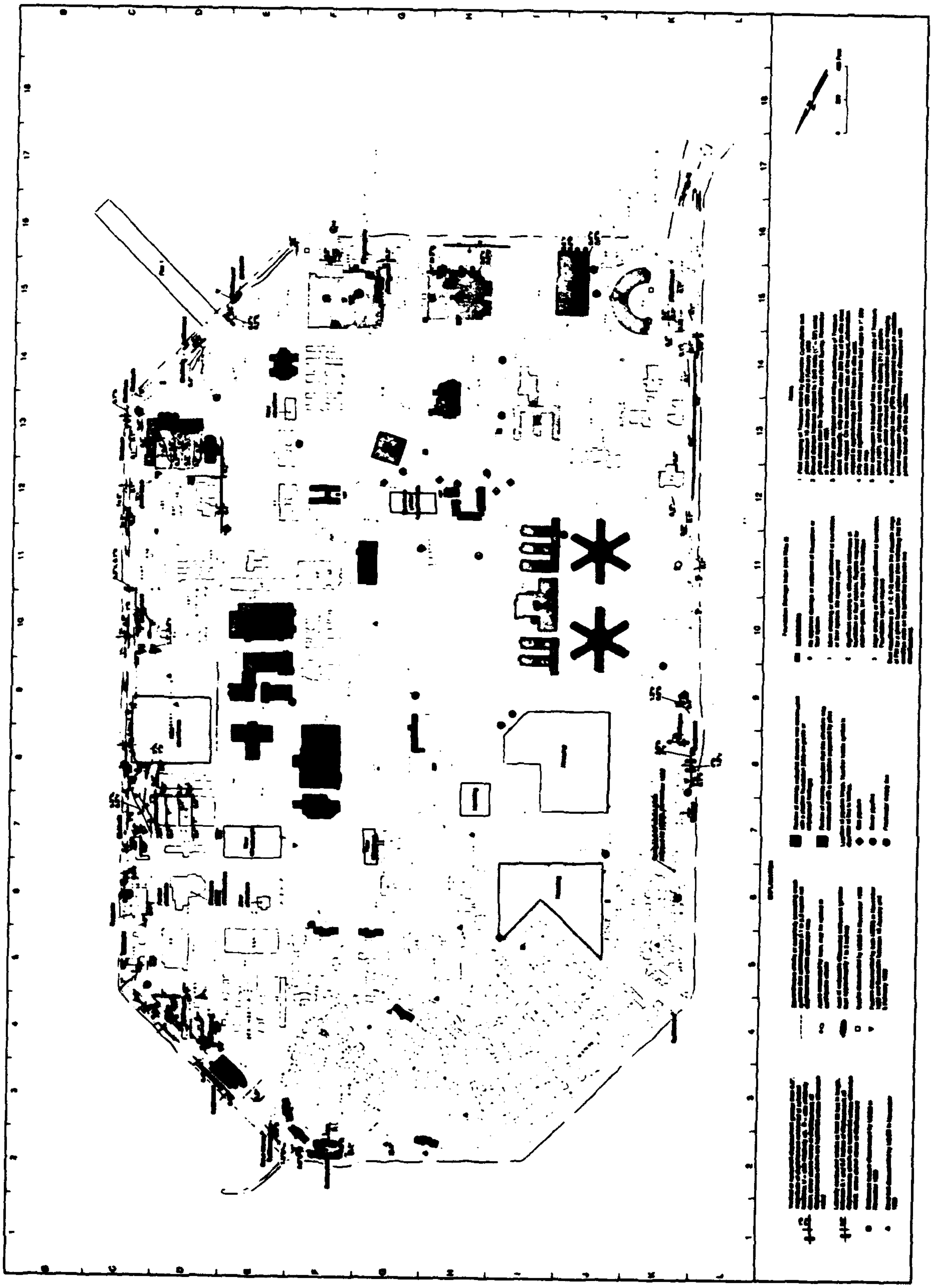
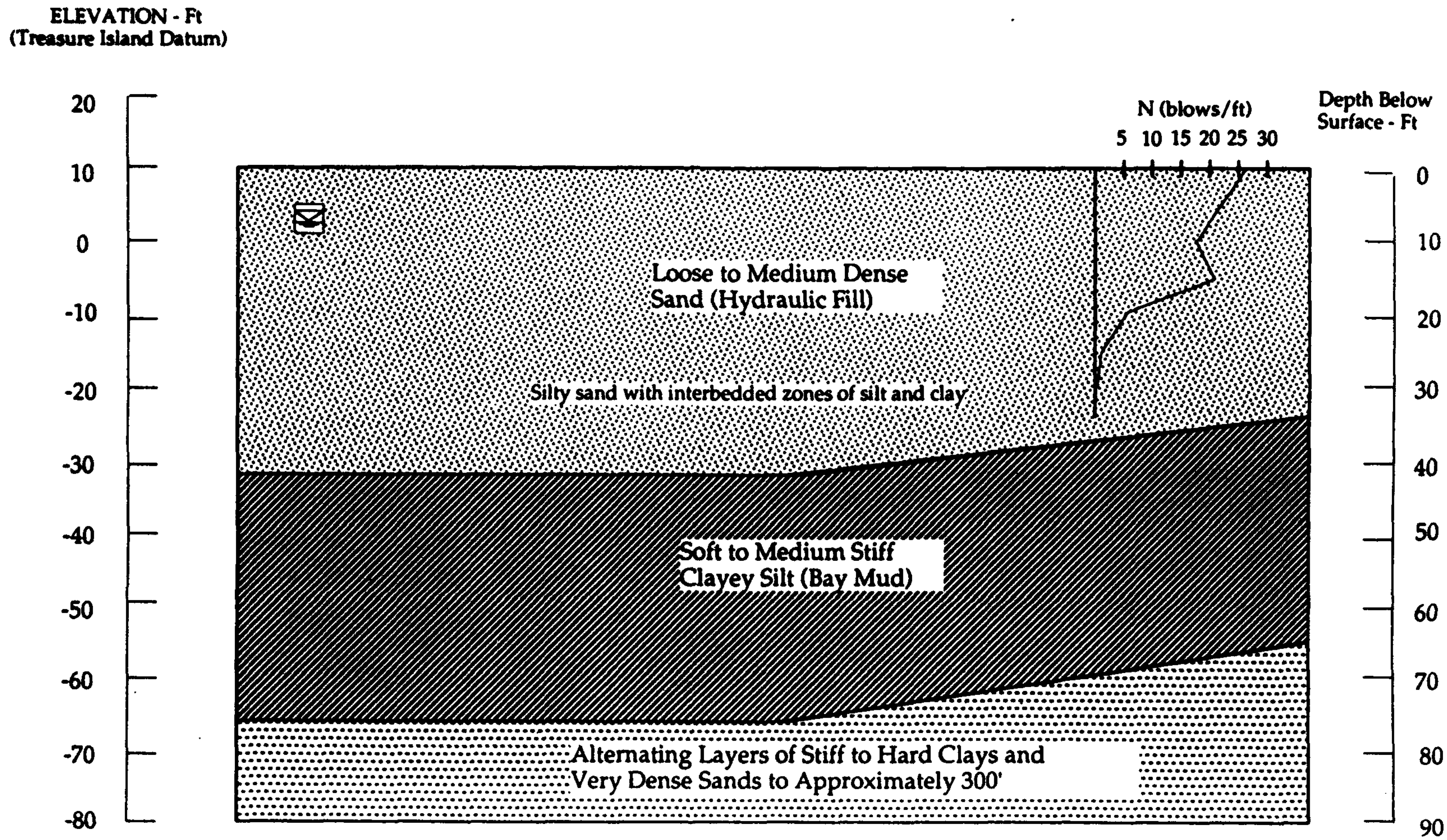


Figure 22. Map of ground deformation and facility distress. (Reference 4)




SCALE: 1" = 30' Horizontal
1" = 20' Vertical

Figure 23. Subsurface profile, Medical/dental Clinic. (Reference 13)

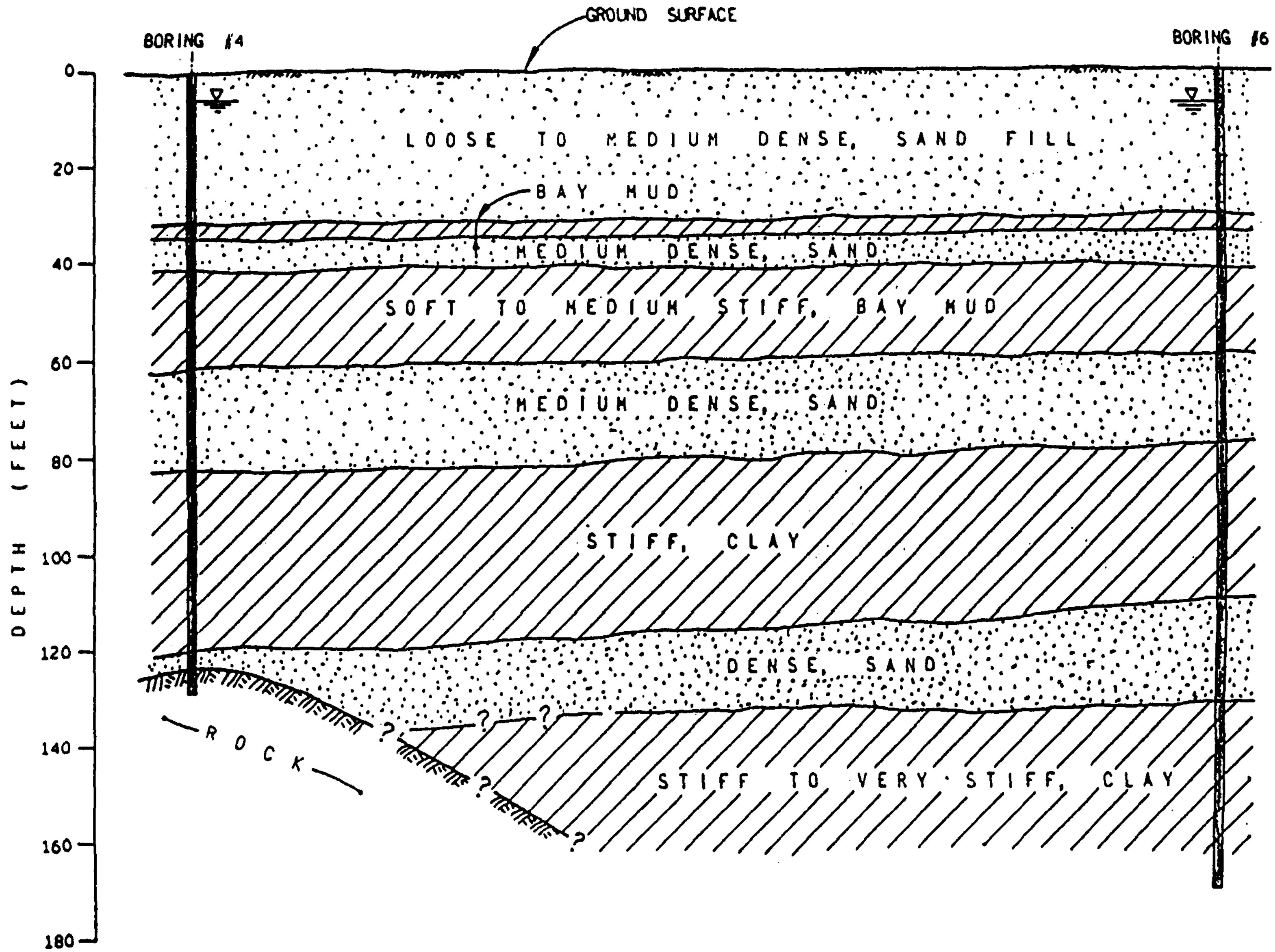
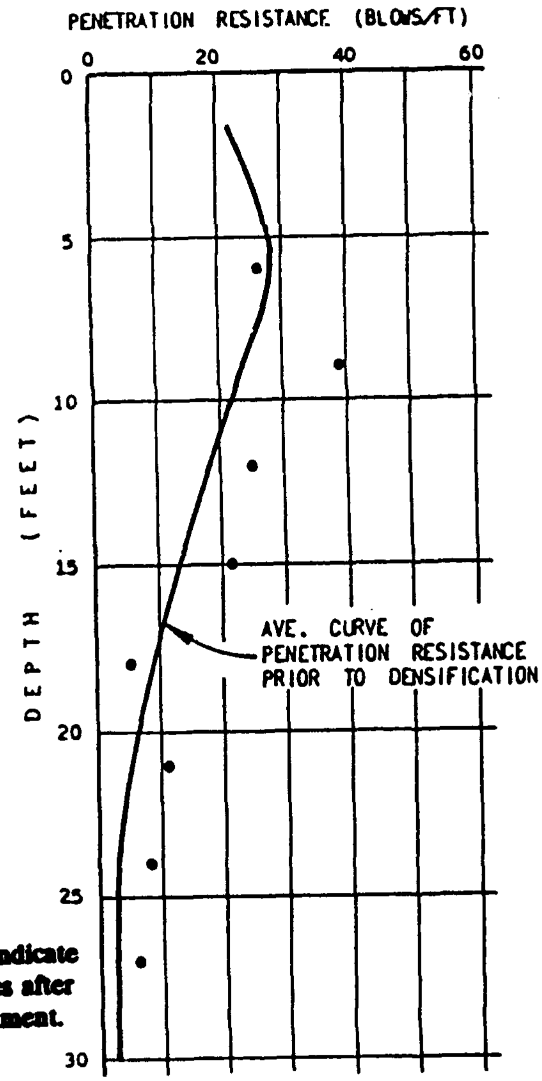



Figure 24. Subsurface profile, Building 450 (Reference 13)




**Note:** **The points indicate SPT N-values after ground treatment.**

Figure 25. Standard Penetration Test, Building 450. (Reference 13)

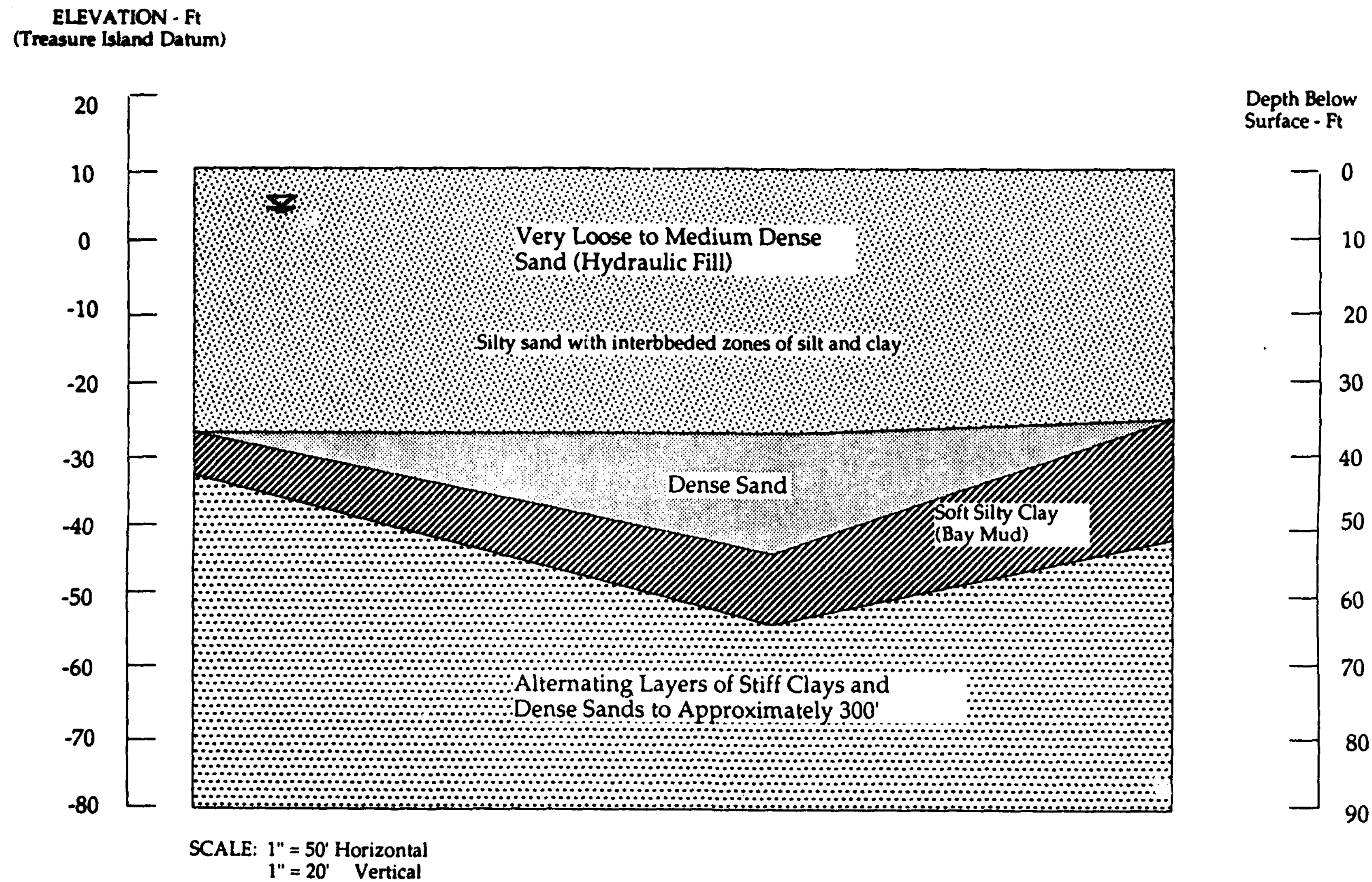



Figure 26. Subsurface profile, Buildings 487, 488 and 489 (Reference 13)

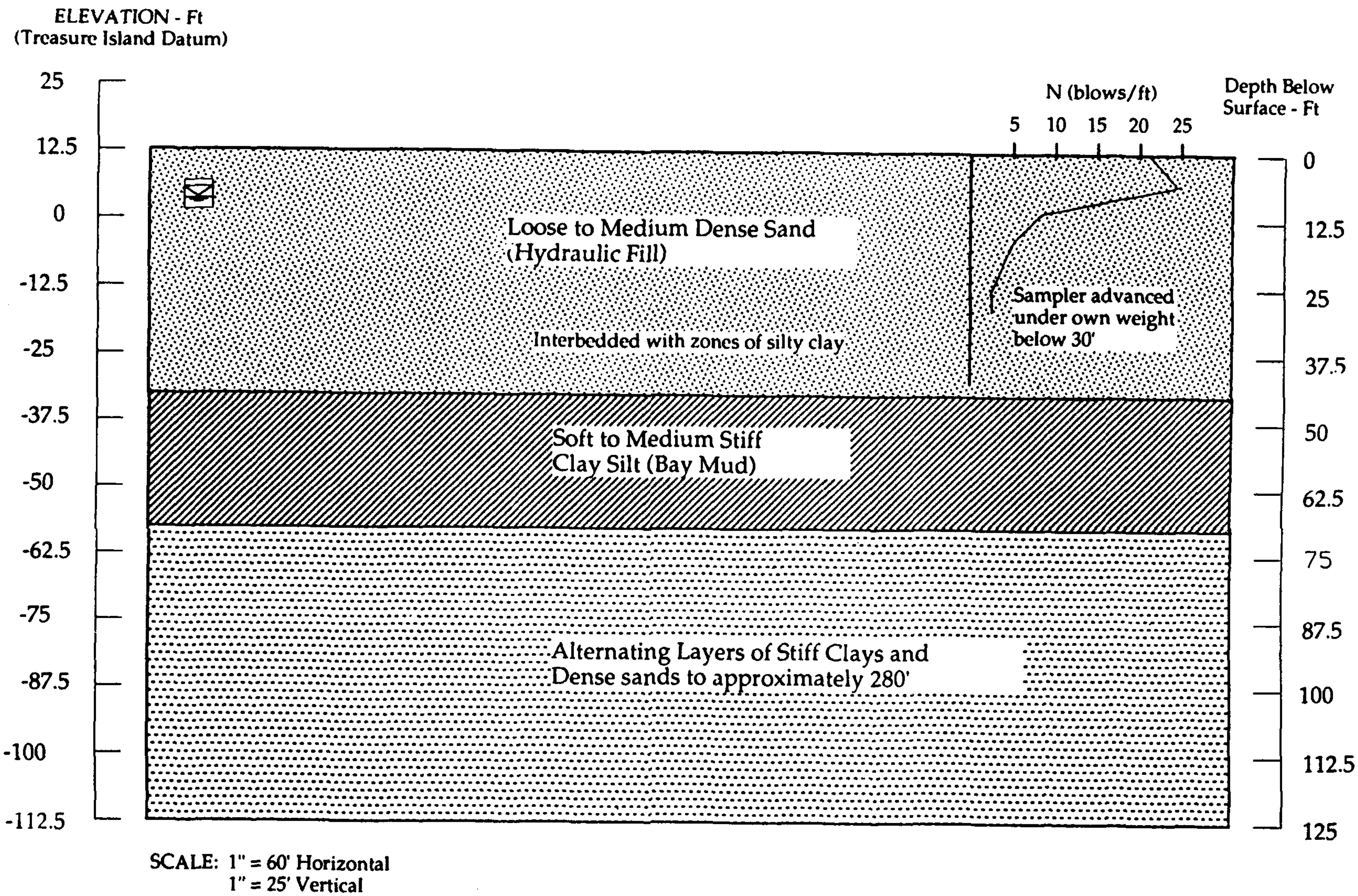



Figure 27. Subsurface profile, Building 453. (Reference 13)

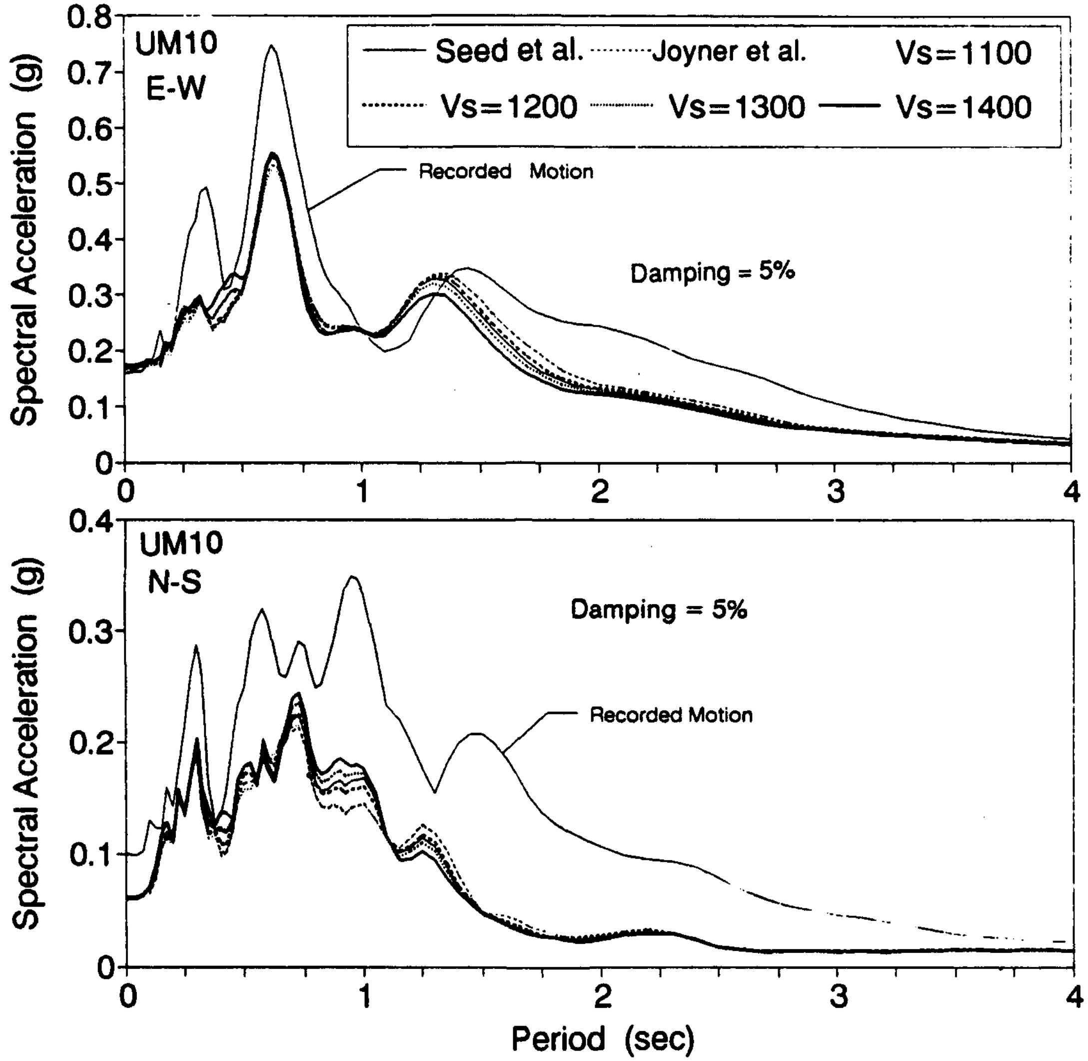



Figure 28. Spectral accelerations using variety of assumptions. (Reference 5)

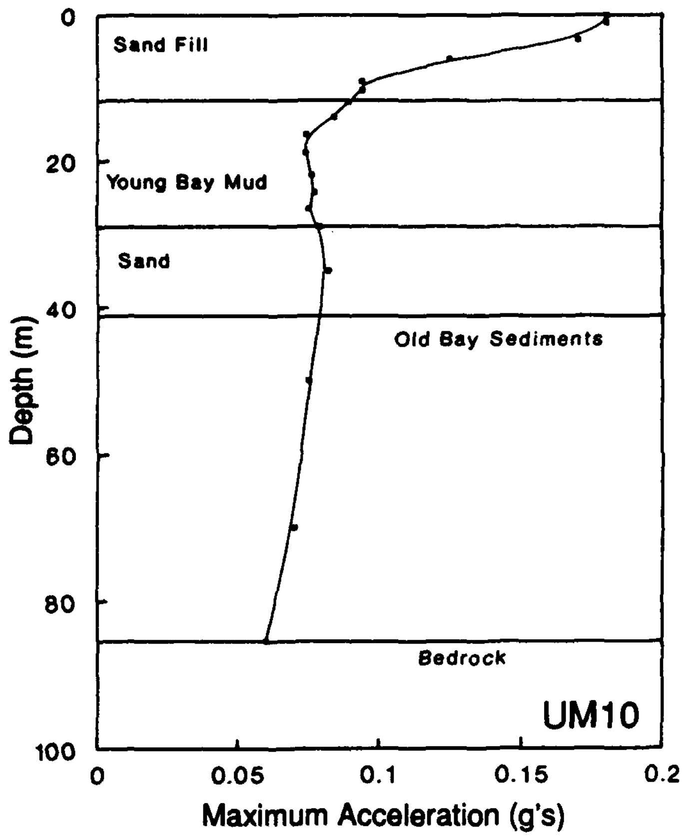



Figure 29. Computed acceleration. (Reference 5)

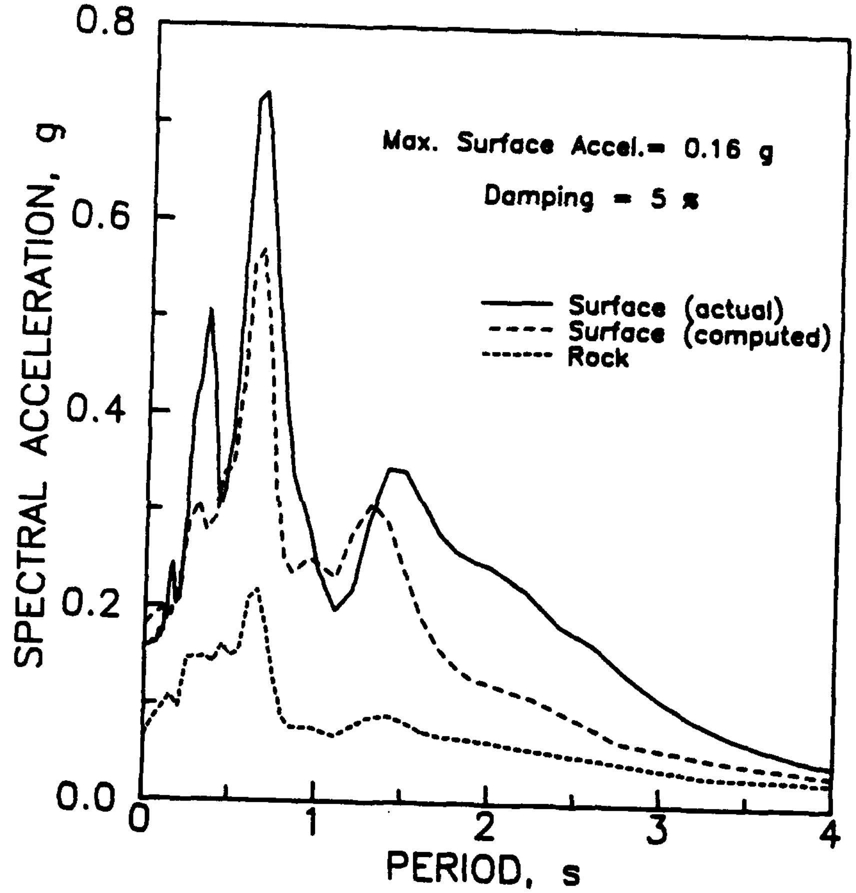



Figure 30. Comparison between calculated and recorded spectra. (Reference 5)

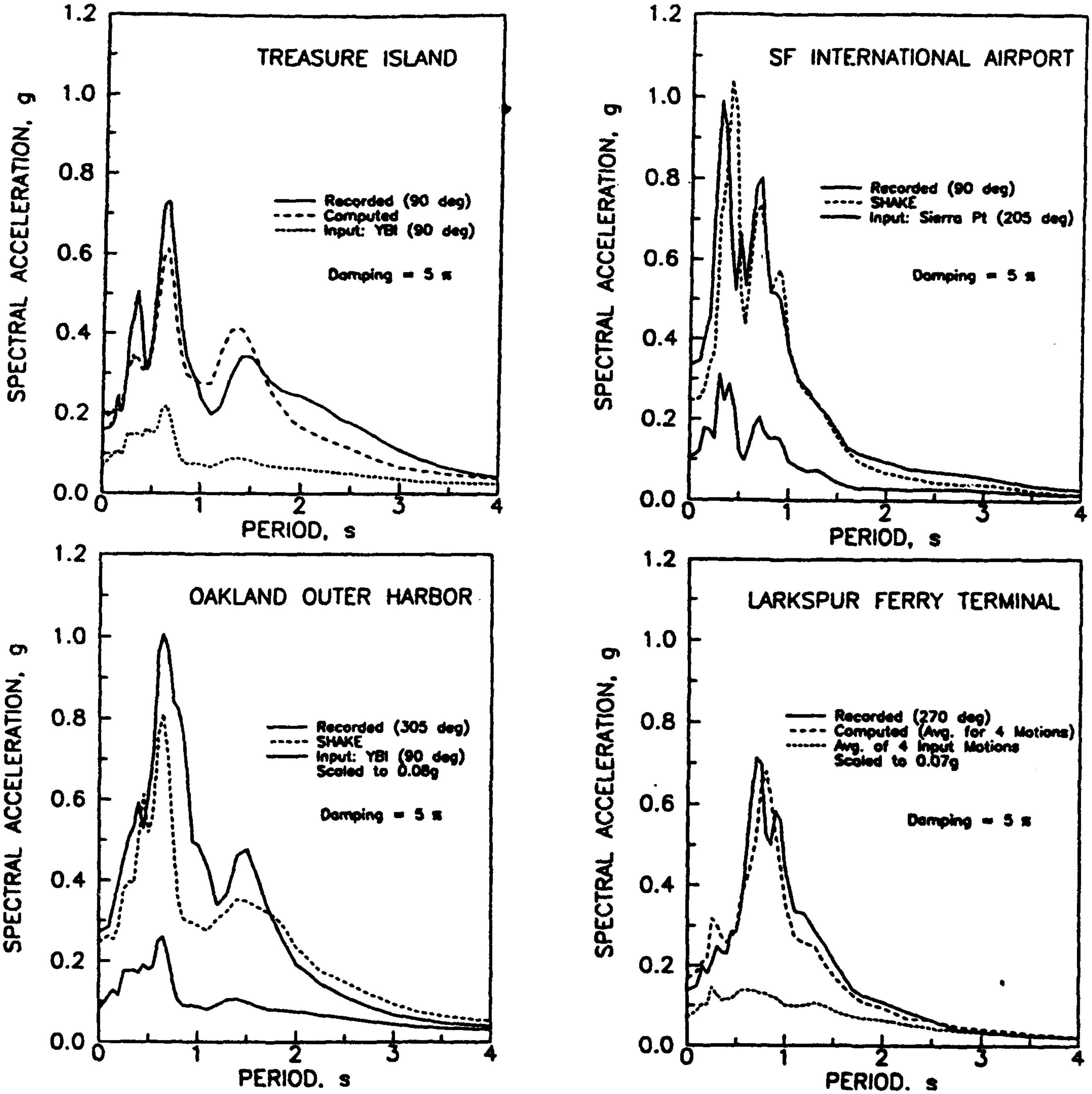



Figure 31. Comparison between calculated and recorded spectra for several sites. (Reference 2)

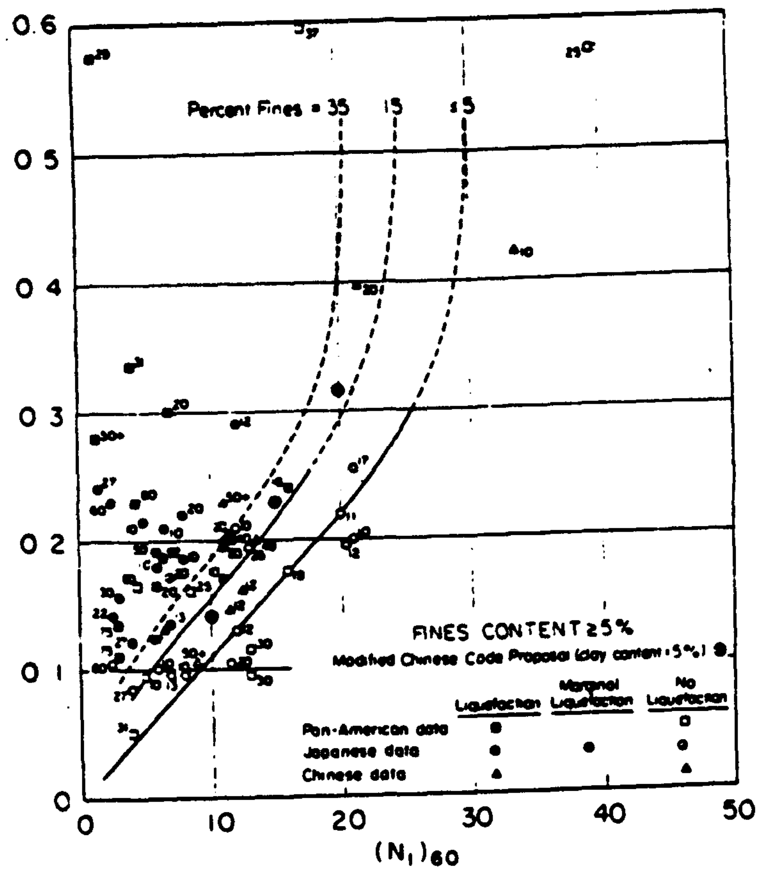



Figure 32. Seed's liquefaction chart. (Reference 21)

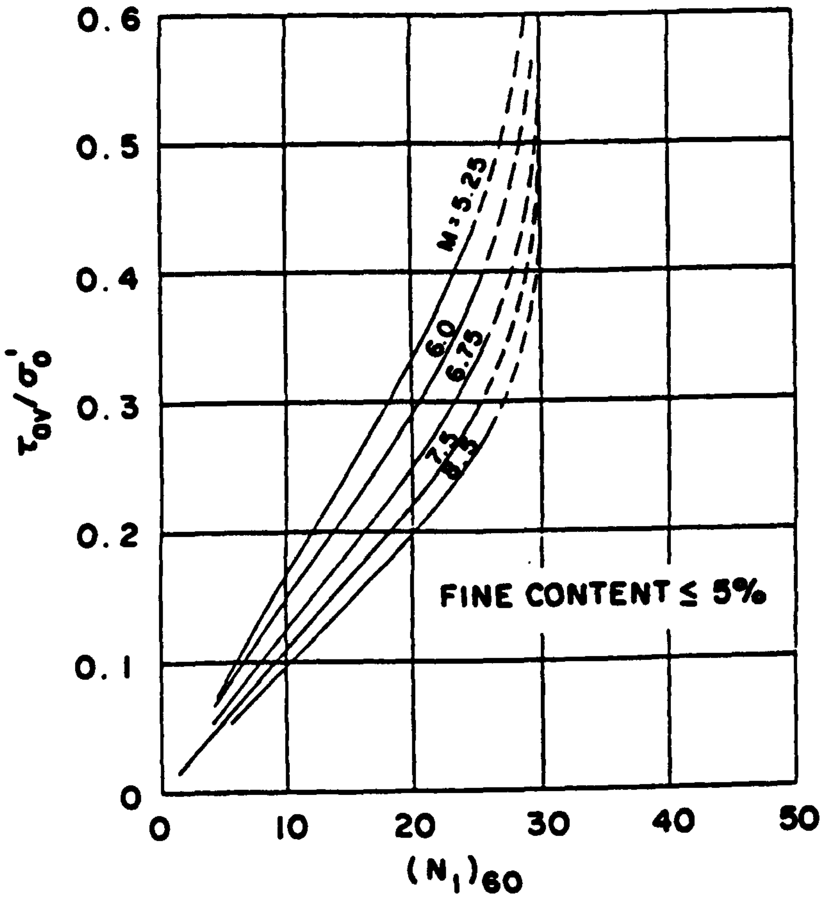



Figure 33. Liquefaction assessment for various magnitudes. (Reference 20)

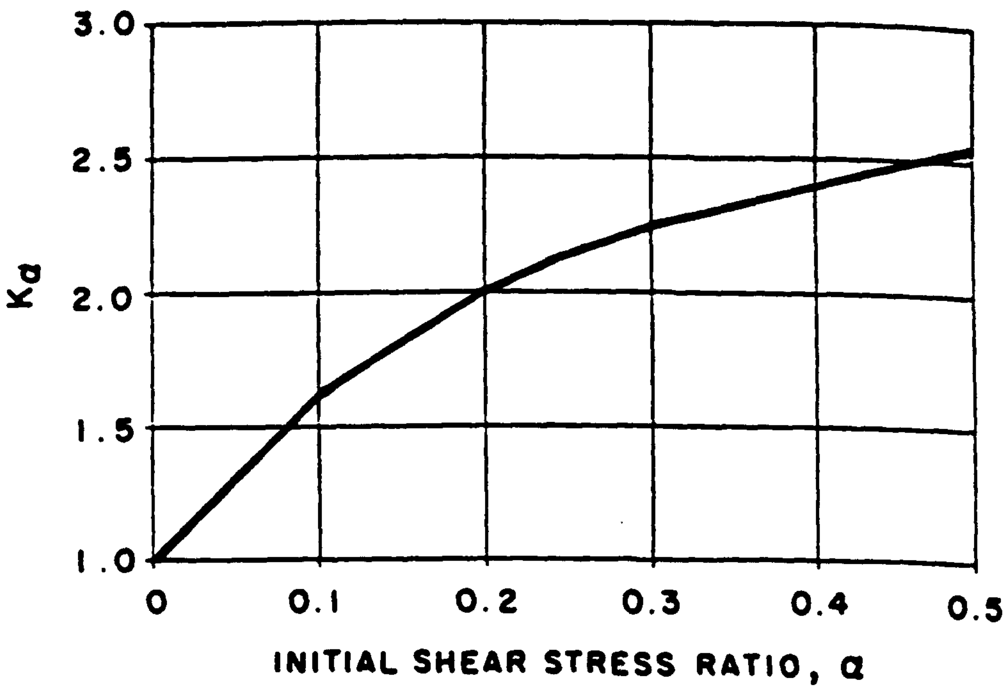



Figure 34. Folsom gravel Kα (Reference 17)

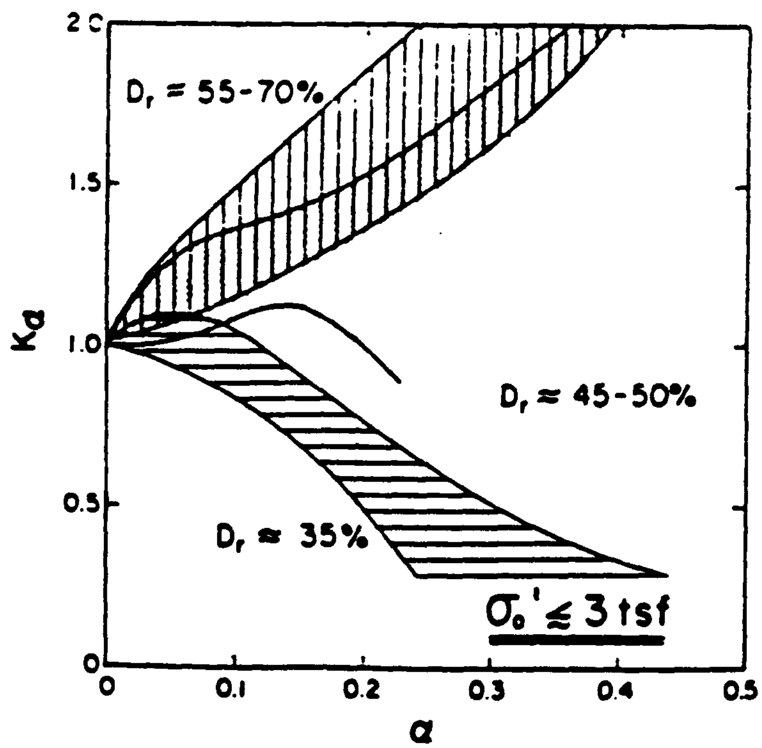



Figure 35. Variation with relative density. (Reference 20)

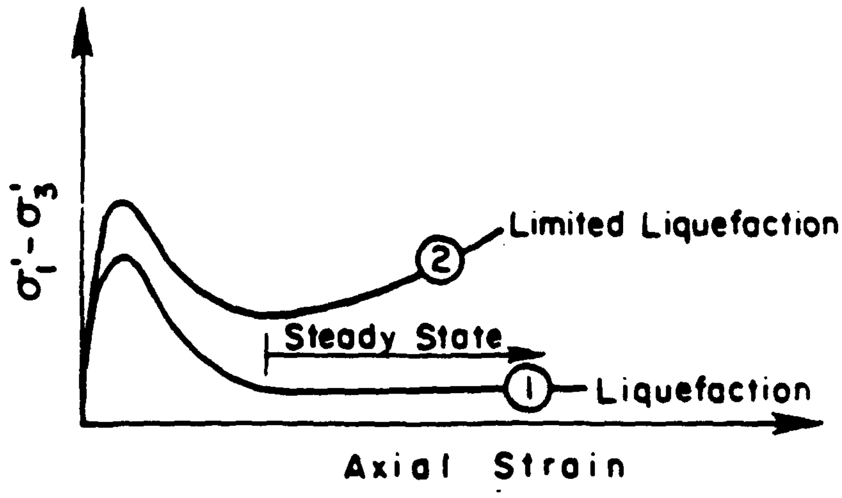



Figure 36 Types of contractive deformation. (Reference 20)

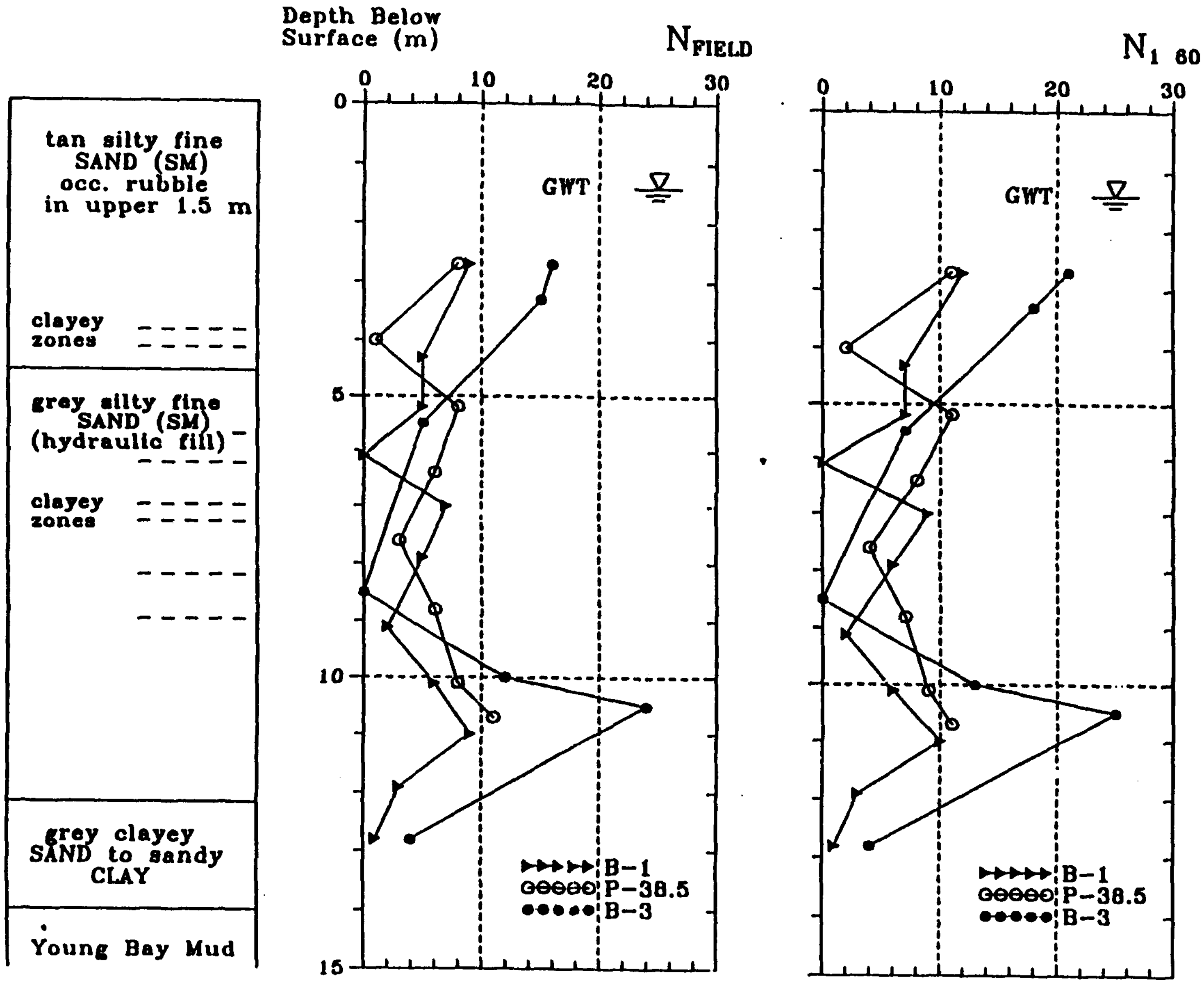



Figure 37. Standard Penetration Test. (Reference 23)

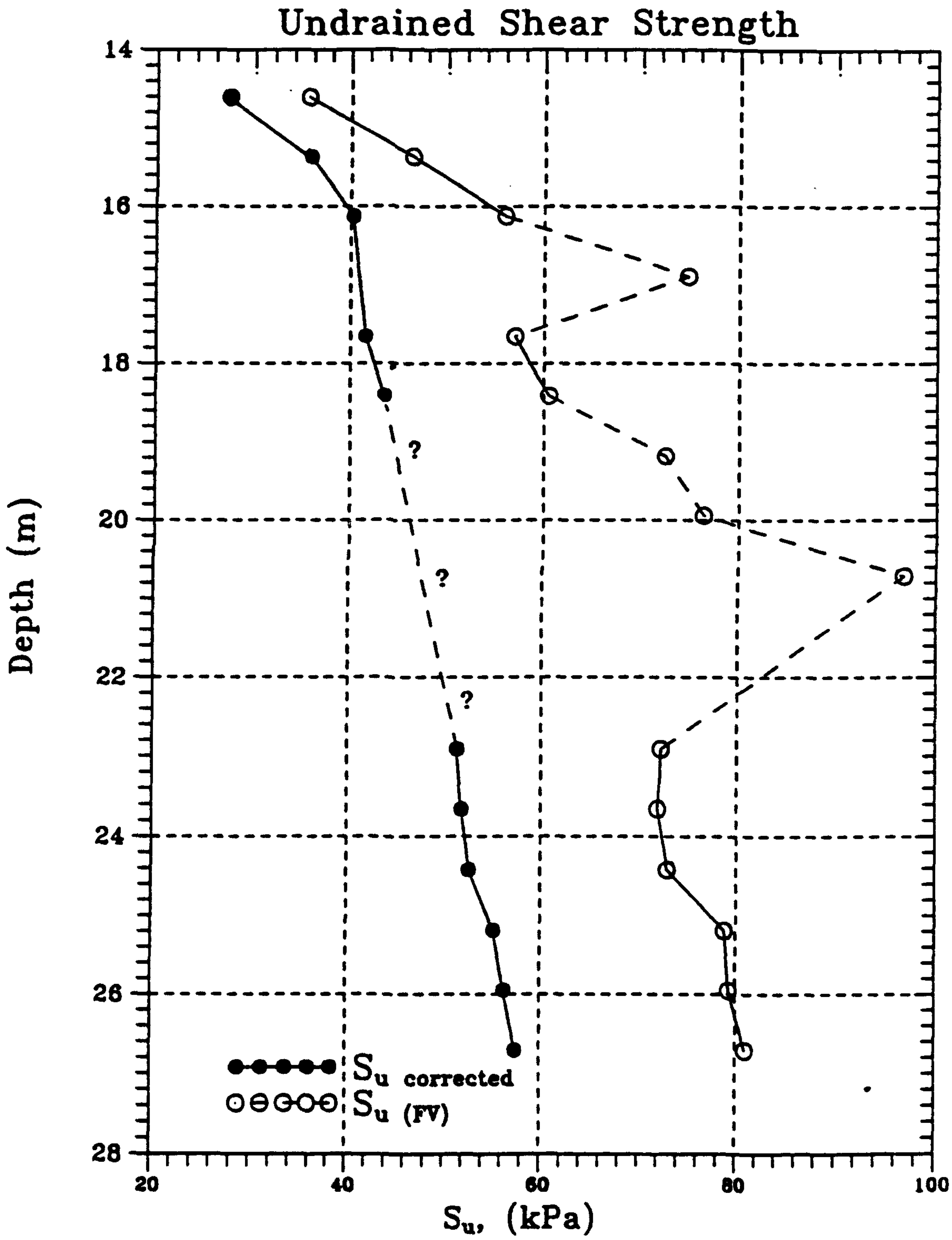



Figure 38. Undrained shear strength. (Reference 23)

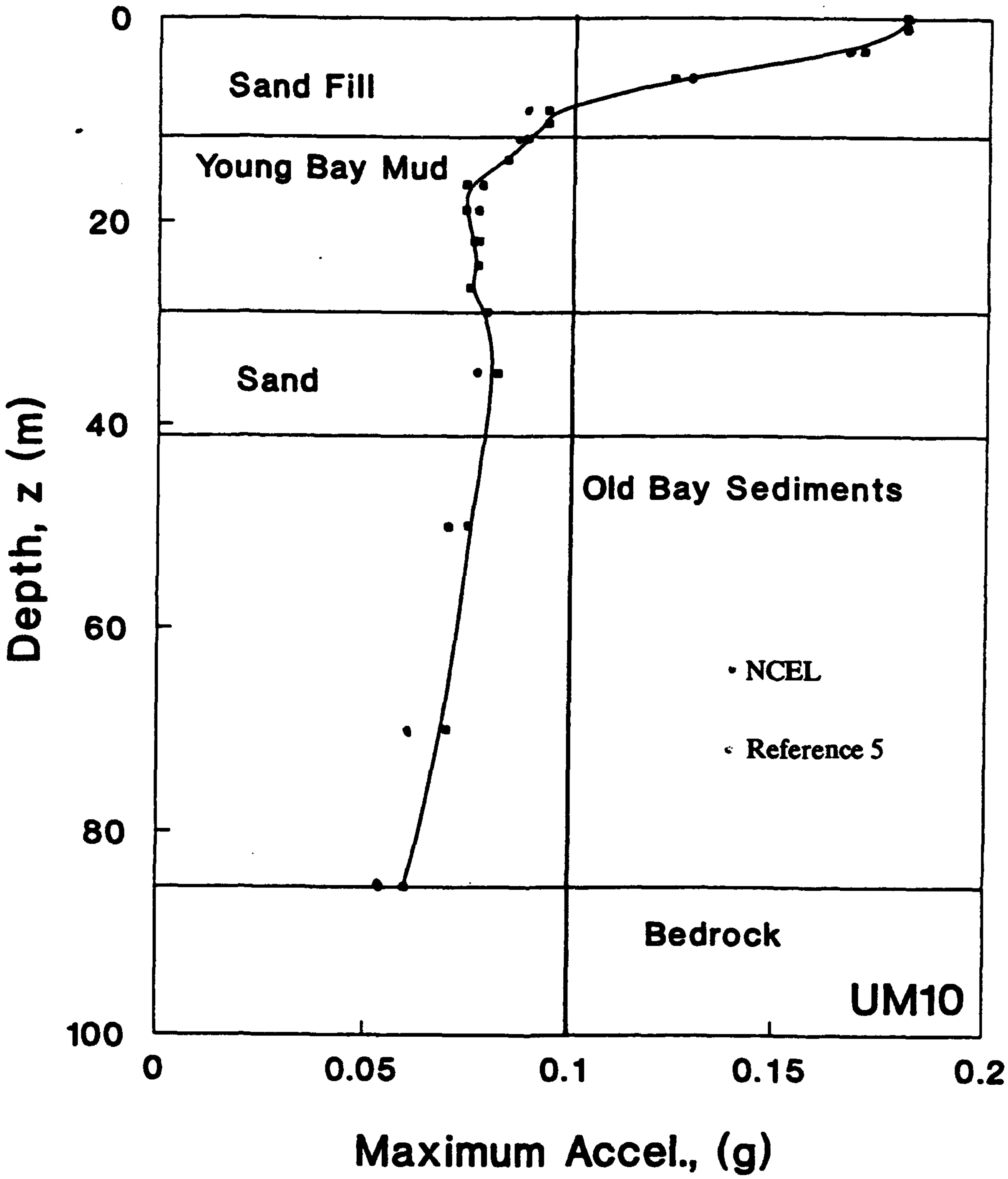



Figure 39 NCEL computed results shown with results from Reference 5.

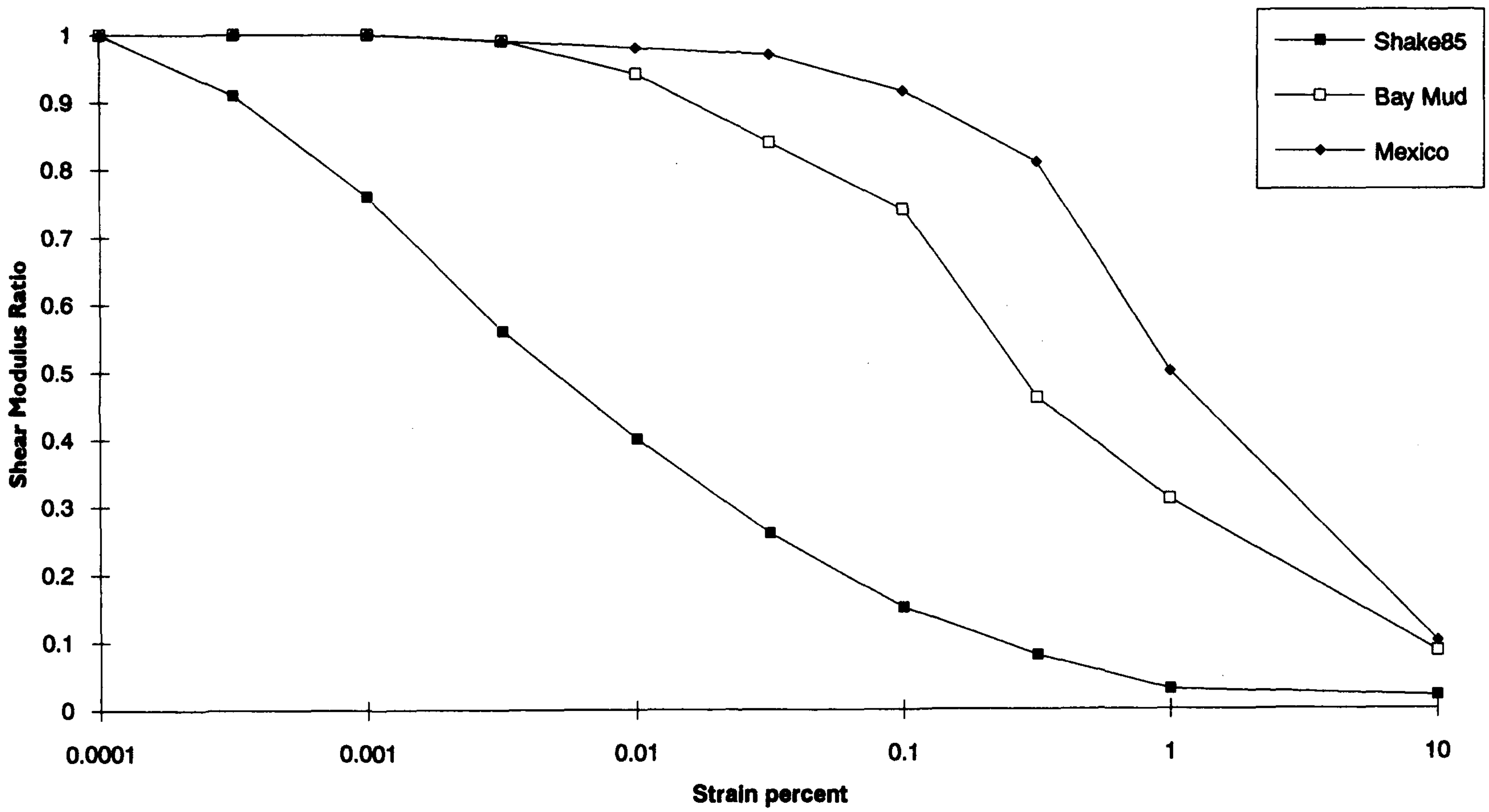



Figure 40. Shear modulus as function of strain.

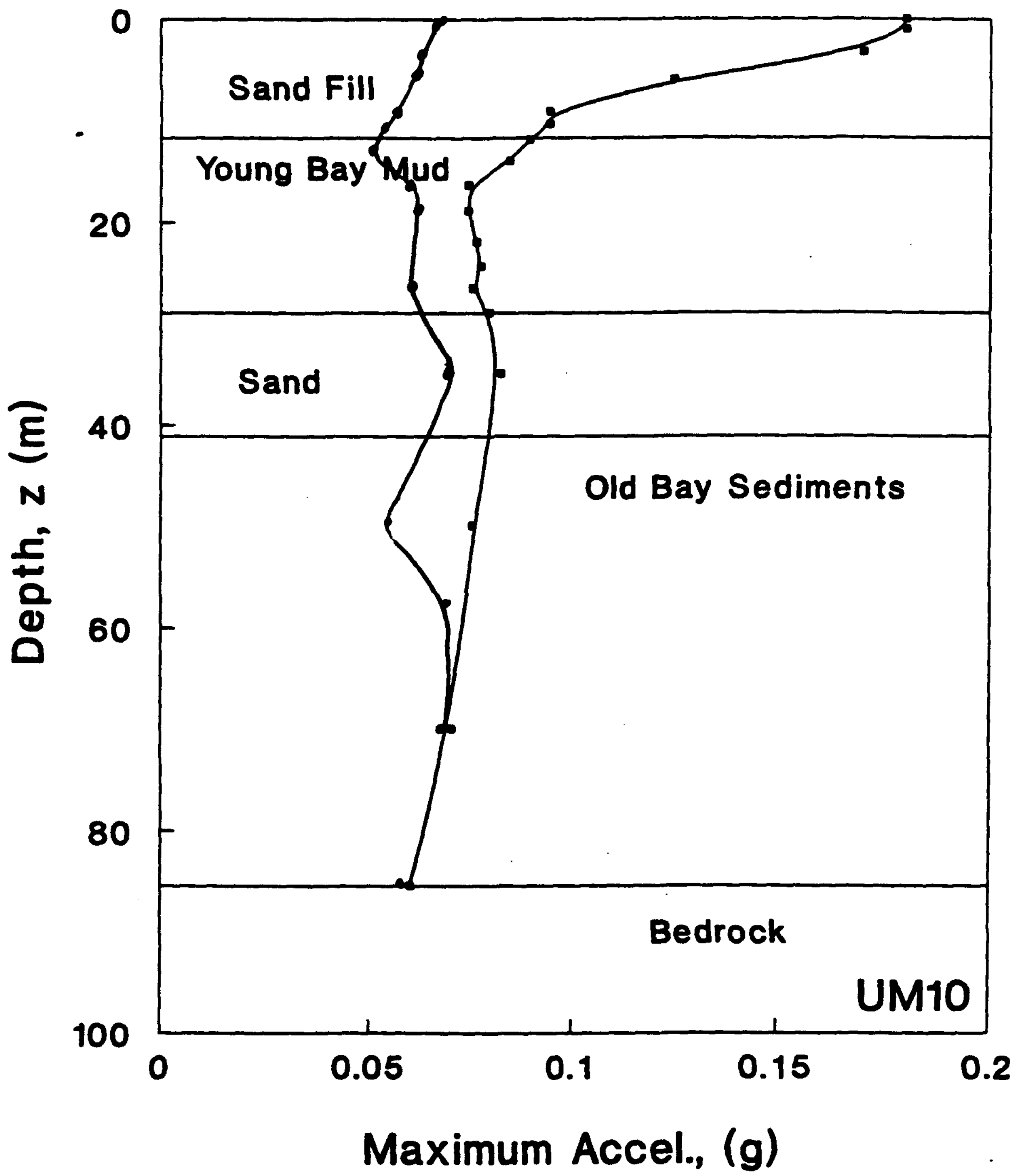



Figure 41. NCEL computed results without amplification compared with Figure 39.

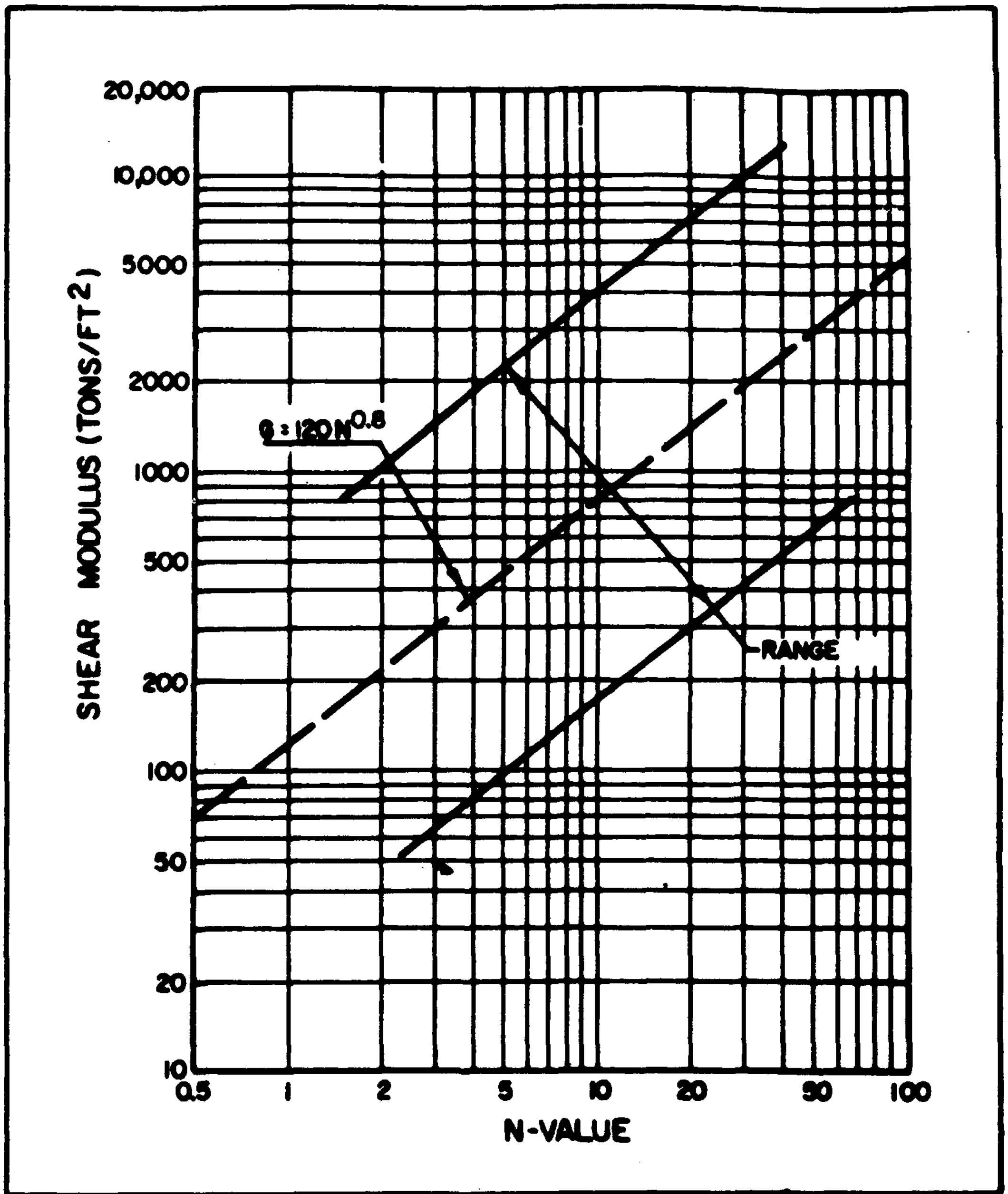



Figure 42. Shear modulus as function of blowcount.

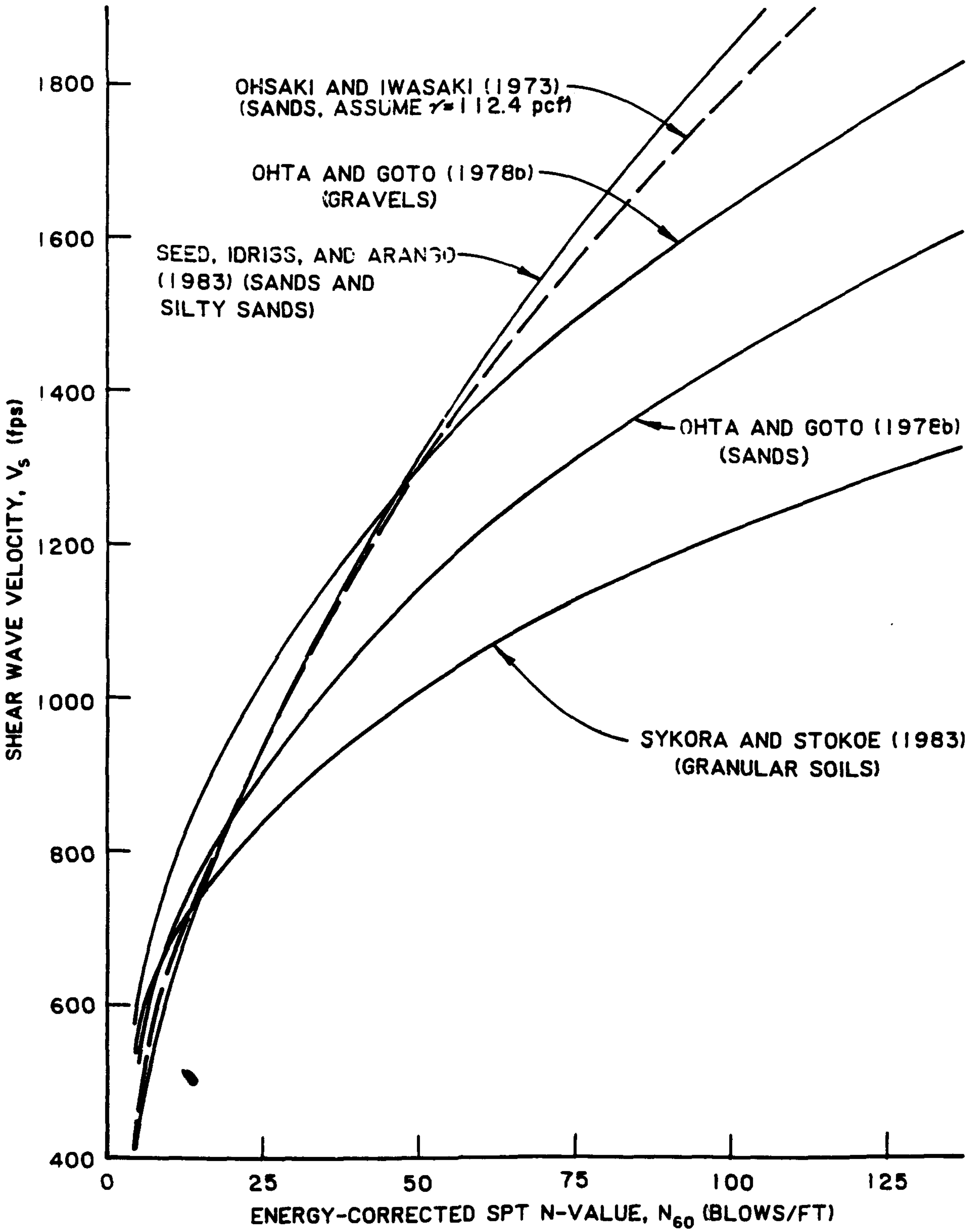



Figure 43. Shear wave velocity as function of blowcount. (Reference 29)

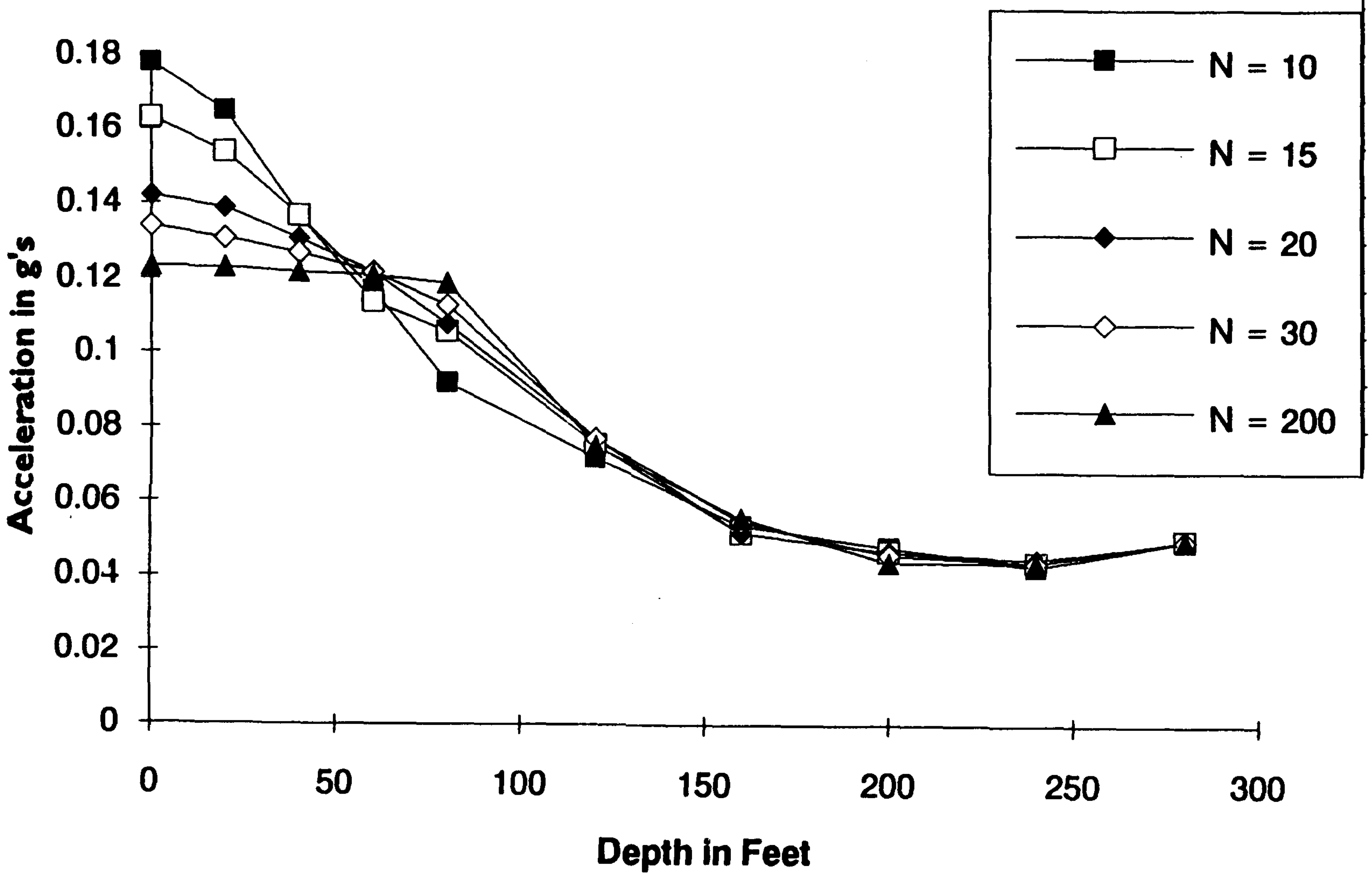



Figure 44. Effects of sand fill as function of blowcount.

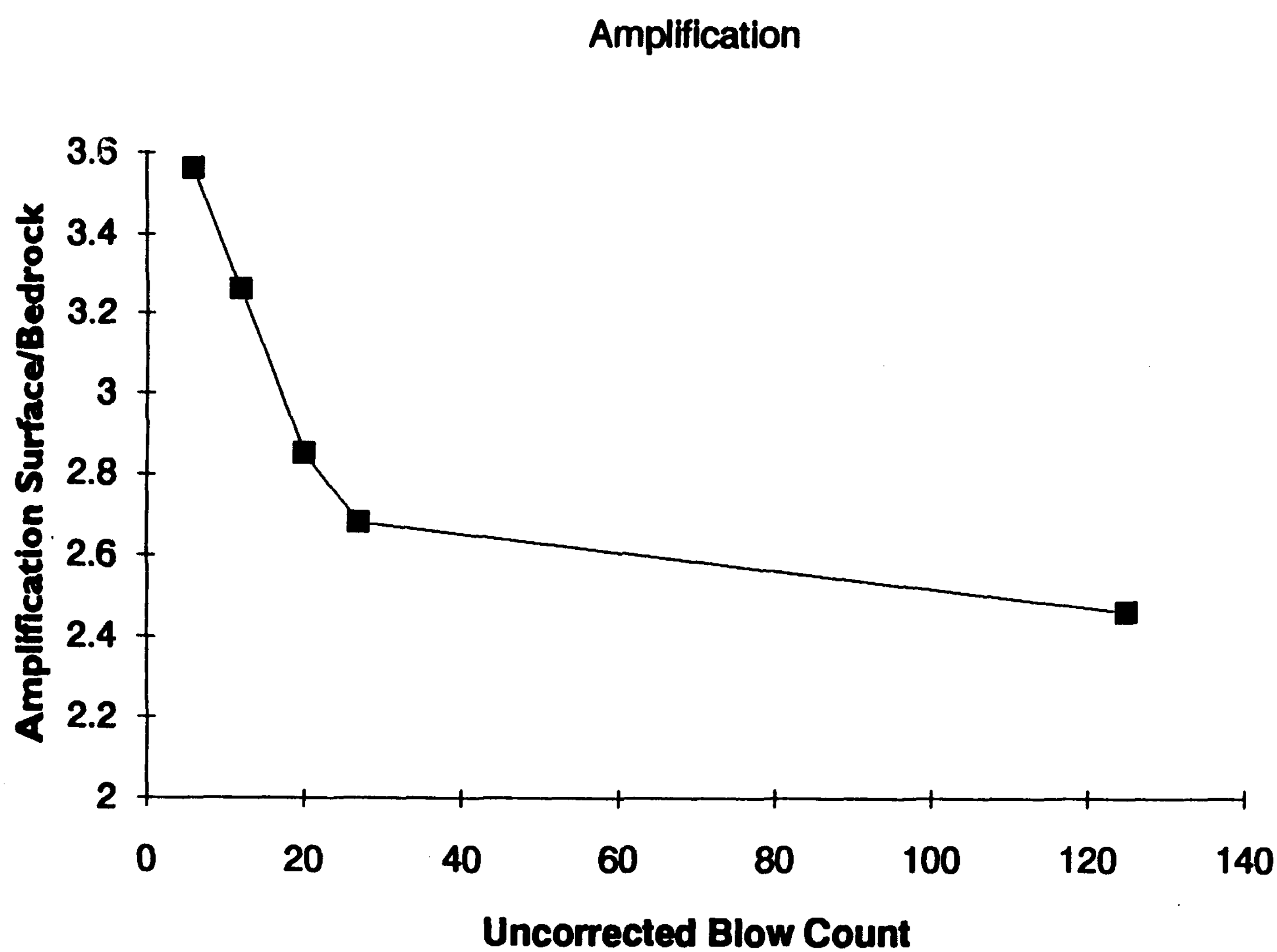



Figure 45. Amplification as function of blowcount.

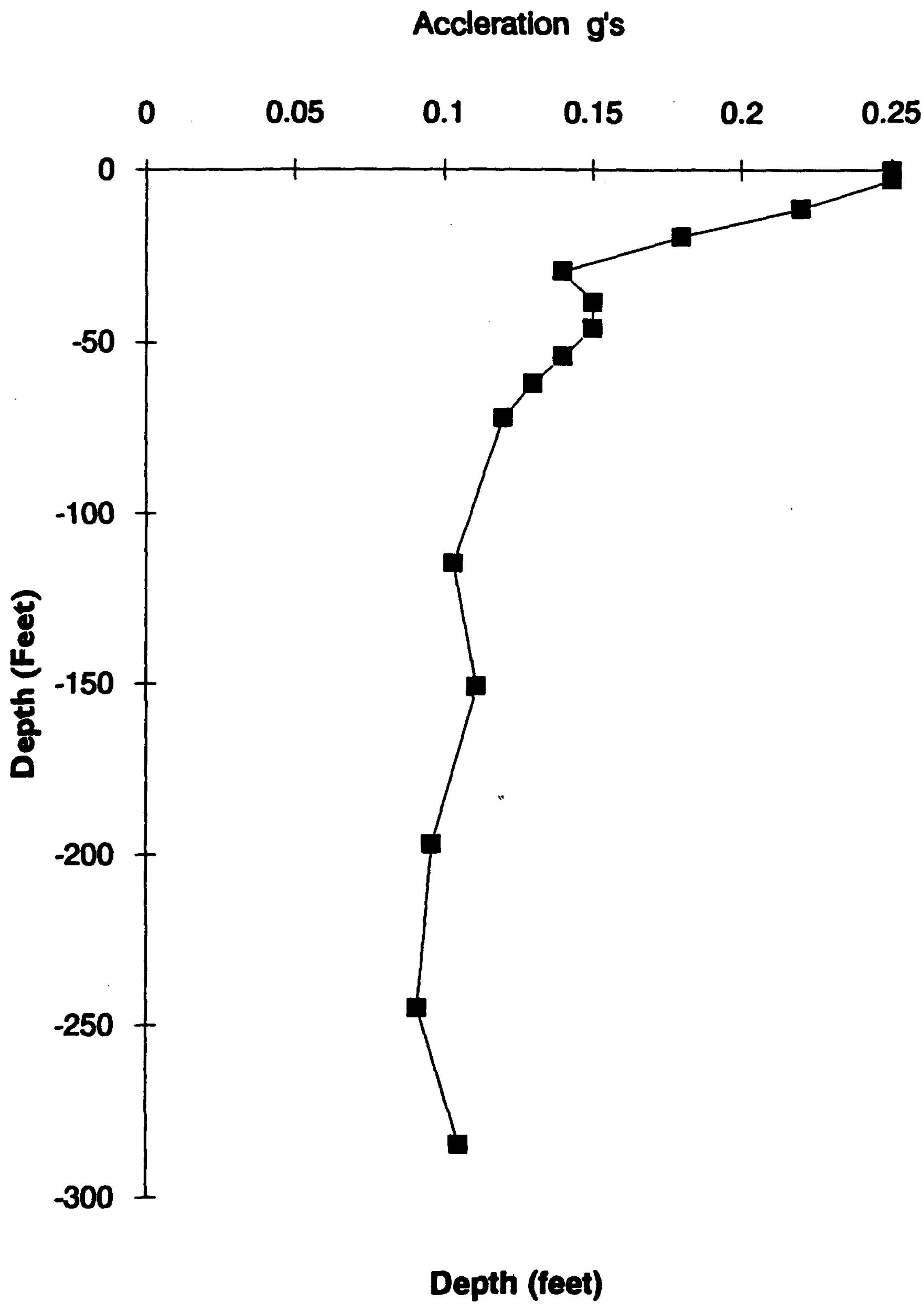



Figure 46. Acceleration for an event 2 times the Loma Prieta event.

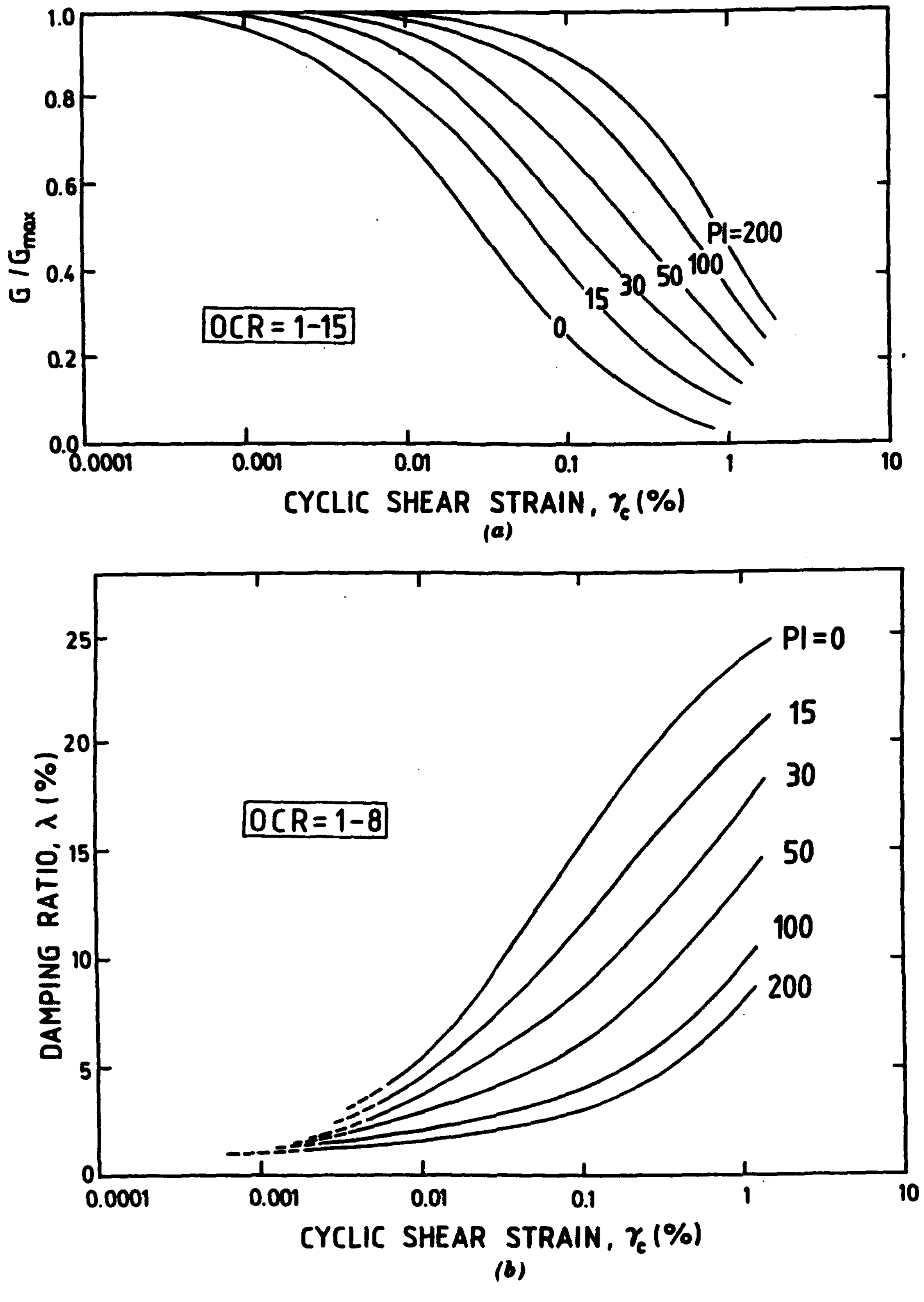



Figure 47. Effect of soil plasticity.

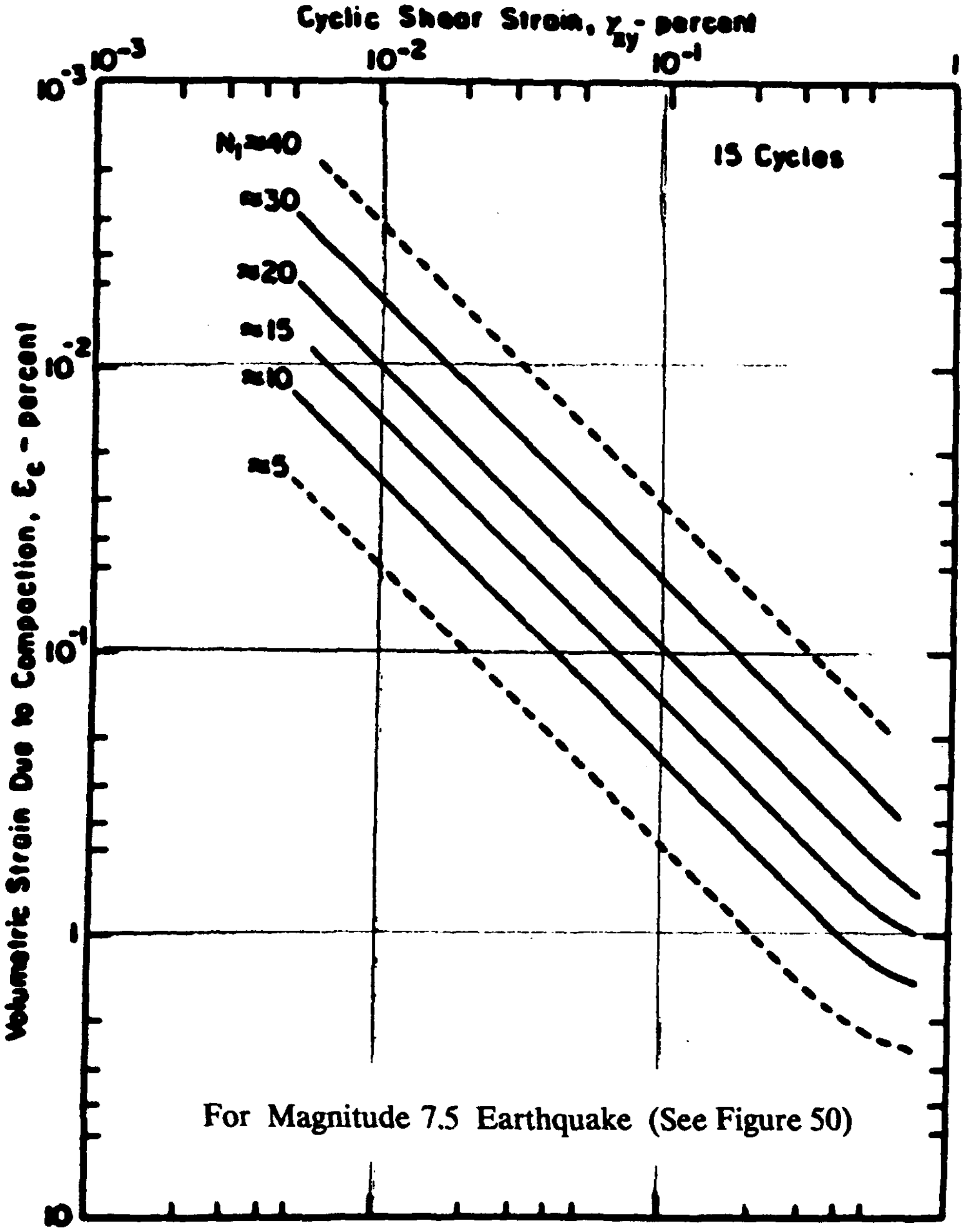



Figure 48. Prediction of volume strain from cyclic shear strain. (Reference 32)

| Earthquake magnitude (1) | Number of representative cycles at 0.65 $\tau_{max}$ (2) | Volumetric strain ratio, $\epsilon_{C,N}/\epsilon_{C,N=15}$ (3) |
|---|---|---|
| 8-1/2 | 26 | 1.25 |
| 7-1/2 | 15 | 1.0 |
| 6-3/4 | 10 | 0.85 |
| 6 | 5 | 0.6 |
| 5-1/4 | 2–3 | 0.4 |

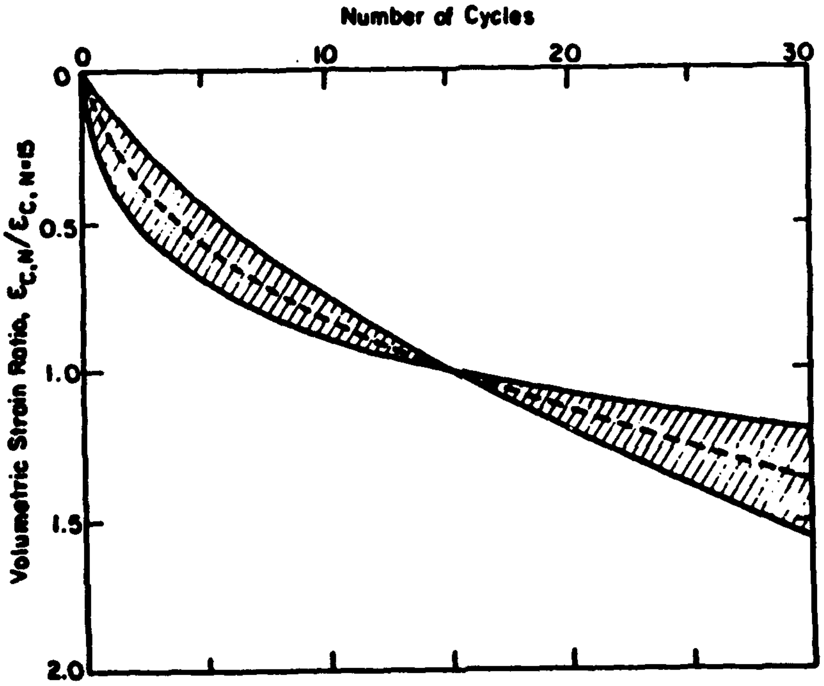



Figure 49. Adjustment factors for other magnitudes for use with Figure 49 (Reference 32).

**TABLE 5.—Computation of Settlement for Deposit of Dry Sand**

| Layer number (1) | Thickness (ft) (2) | $\sigma_o = \sigma'_o$ (psf) (3) | $D_r$ (%) (4) | $N_1$ (5) | $G_{max}$[a] (ksf) (6) | $\gamma_{eff}(G_{eff}/G_{max})$ (7) | $\gamma_{eff}$ (8) | $\epsilon_{C,M=7.5}$ (%) (9) | $\epsilon_{C,M=6.6}$[b] (%) (10) | $2\epsilon_{C,M=6.6}$[c] (%) (11) | Settlement (in.) (12) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 5 | 240 | 45 | 9 | 520 | $1.3 \times 10^{-4}$ | $5 \times 10^{-4}$ | 0.14 | 0.11 | 0.22 | 0.13 |
| 2 | 5 | 714 | 45 | 9 | 900 | 2.3 | 8 | 0.23 | 0.18 | 0.36 | 0.22 |
| 3 | 10 | 1,425 | 45 | 9 | 1,270 | 3.2 | 12 | 0.35 | 0.28 | 0.56 | 0.67 |
| 4 | 10 | 2,375 | 45 | 9 | 1,630 | 4.0 | 14 | 0.40 | 0.32 | 0.64 | 0.77 |
| 5 | 10 | 3,325 | 45 | 9 | 1,930 | 4.5 | 15 | 0.45 | 0.36 | 0.72 | 0.86 |
| 6 | 10 | 4,275 | 45 | 9 | 2,190 | 4.6 | 13 | 0.38 | 0.30 | 0.60 | 0.72 |
| Total | | | | | | | | | | | 3.37 |

[a] $G_{max} = K_2 \cdot 1{,}000\,(\sigma'_m)^{1/2} = 20\,N_1^{1/3}(\sigma'_m)^{1/2} \times 1{,}000$.
[b] $\epsilon_{C,M=6.6}/\epsilon_{C,M=7.5} = 0.80$.
[c] Multidirectional effect.

Figure 50. Example study from Reference 32.

ITERATION NUMBER 5
THE CALCULATION HAS BEEN CARRIED OUT IN THE TIME DOMAIN WITH EFF. STRAIN = .65* MAX. STRAIN

| LAYER | TYPE | DEPTH | EFF. STRAIN | NEW DAMP. | DAMP USED | ERROR | NEW G | G USED | ERROR |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 1.5 | .00024 | .012 | .012 | -.5 | 8911.661 | 8908.655 | .0 |
| 2 | 2 | 7.1 | .05256 | .122 | .122 | .6 | 177.289 | 178.530 | -.7 |
| 3 | 2 | 15.4 | .10305 | .156 | .153 | 1.9 | 175.674 | 181.773 | -3.5 |
| 4 | 2 | 24.5 | .16819 | .180 | .176 | 2.0 | 150.487 | 157.011 | -4.3 |
| 5 | 2 | 34.0 | .10367 | .156 | .154 | .9 | 295.920 | 299.632 | -1.3 |
| 6 | 1 | 42.3 | .03958 | .070 | .072 | -1.7 | 868.761 | 864.218 | .5 |
| 7 | 1 | 50.0 | .05525 | .078 | .080 | -1.8 | 638.513 | 634.590 | .6 |
| 8 | 1 | 58.0 | .05797 | .079 | .080 | -1.1 | 635.270 | 632.799 | .4 |
| 9 | 1 | 67.0 | .02061 | .058 | .059 | -.7 | 1897.790 | 1892.720 | .3 |
| 10 | 1 | 83.5 | .05293 | .077 | .078 | -.3 | 842.993 | 842.105 | .1 |
| 11 | 2 | 105.0 | .04596 | .116 | .114 | 1.9 | 1164.357 | 1188.113 | -2.0 |
| 12 | 2 | 125.0 | .04895 | .119 | .118 | .8 | 1243.185 | 1254.658 | -.9 |
| 13 | 1 | 143.0 | .02247 | .060 | .060 | -.7 | 2861.900 | 2853.648 | .3 |
| 14 | 1 | 162.7 | .02054 | .058 | .059 | -.8 | 3332.309 | 3321.688 | .3 |
| 15 | 1 | 186.0 | .02210 | .060 | .060 | -.7 | 3307.761 | 3298.039 | .3 |
| 16 | 1 | 209.3 | .02343 | .060 | .061 | -.6 | 3288.130 | 3280.215 | .2 |
| 17 | 1 | 233.0 | .01680 | .055 | .055 | -.1 | 5089.466 | 5088.509 | .0 |
| 18 | 2 | 252.5 | .04534 | .115 | .114 | 1.5 | 2119.916 | 2152.447 | -1.5 |
| 19 | 1 | 272.5 | .01988 | .058 | .058 | -.3 | 5004.967 | 4999.303 | .1 |

VALUES IN TIME DOMAIN

| LAYER | TYPE | THICKNESS FT | DEPTH FT | MAX STRAIN PRCNT | MAX STRESS PSF | TIME SEC |
|---|---|---|---|---|---|---|
| 1 | 2 | 3.0 | 1.5 | .00037 | 33.02 | 11.84 |
| 2 | 2 | 8.3 | 7.1 | .08086 | 143.36 | 11.86 |
| 3 | 2 | 8.3 | 15.4 | .15854 | 278.52 | 11.88 |
| 4 | 2 | 10.0 | 24.5 | .25875 | 389.38 | 11.88 |
| 5 | 2 | 9.0 | 34.0 | .15950 | 471.98 | 11.88 |
| 6 | 1 | 7.5 | 42.3 | .06089 | 529.00 | 11.86 |
| 7 | 1 | 8.0 | 50.0 | .08500 | 542.76 | 11.86 |
| 8 | 1 | 8.0 | 58.0 | .08919 | 566.61 | 11.92 |
| 9 | 1 | 10.0 | 67.0 | .03171 | 601.73 | 11.94 |
| 10 | 1 | 23.0 | 83.5 | .08143 | 686.48 | 11.98 |
| 11 | 2 | 20.0 | 105.0 | .07071 | 823.30 | 12.04 |
| 12 | 2 | 20.0 | 125.0 | .07531 | 936.24 | 12.02 |
| 13 | 1 | 16.0 | 143.0 | .03457 | 989.42 | 12.00 |
| 14 | 1 | 23.3 | 162.7 | .03160 | 1053.17 | 11.98 |
| 15 | 1 | 23.3 | 186.0 | .03400 | 1124.75 | 11.96 |
| 16 | 1 | 23.3 | 209.3 | .03605 | 1185.43 | 11.94 |
| 17 | 1 | 24.0 | 233.0 | .02584 | 1315.28 | 11.26 |
| 18 | 2 | 15.0 | 252.5 | .06975 | 1478.73 | 11.24 |
| 19 | 1 | 25.0 | 272.5 | .03058 | 1530.38 | 11.22 |

Figure 51. SHAKE analysis results of Treasure Island site Loma Prieta earthquake.

```
                                                                      READY
D28   6.6
      A          B          C          D          E          F
1                Tokimatsu Seed Liquefaction Settlement
2                ===================================
3
4   Layer Thickness BlowCount  Eff Shear   Vol Strain Settlement
5   ----- --- ft -- --------- --- % ----  --- % ---- --- In ---
6     1      5.00      9.00   0.0500000   0.1460495     0.18
7     2      5.00      9.00   0.0800000   0.2336792     0.28
8     3     10.00      9.00   0.1200000   0.3505188     0.84
9     4     10.00      9.00   0.1400000   0.4089387     0.98
10    5     10.00      9.00   0.1500000   0.4381486     1.05
11    6     10.00      9.00   0.1300000   0.3797288     0.91
12    7      0.00      0.00   0.0000000   0.0000000     0.00
13    8      0.00      0.00   0.0000000   0.0000000     0.00
14    9      0.00      0.00   0.0000000   0.0000000     0.00
15   10      0.00      0.00   0.0000000   0.0000000     0.00
16   11      0.00      0.00   0.0000000   0.0000000     0.00
17   12      0.00      0.00   0.0000000   0.0000000     0.00
18   13      0.00      0.00   0.0000000   0.0000000     0.00
19   14      0.00      0.00   0.0000000   0.0000000     0.00
20   15      0.00      0.00   0.0000000   0.0000000     0.00
21   16      0.00      0.00   0.0000000   0.0000000     0.00
22   17      0.00      0.00   0.0000000   0.0000000     0.00
23   18      0.00      0.00   0.0000000   0.0000000     0.00
24   19      0.00      0.00   0.0000000   0.0000000     0.00
25   20      0.00      0.00   0.0000000   0.0000000     0.00
26
27
28  Enter earthquake magnitude      6.60
29
30
31
32                Total settlement in inches            3.22
33
34   Enter data in columns B C and D for each sand layer
35   Column B  Layer thickness in Feet
36   Column C  Blowcounts N sub I
37   Column D  Effective Shear Strain in percent
38
39
What would like to do? P(rint), S(ave File), G(et File), Q(uit)
```

Figure 52. LIQSS results for example in Figure 50.

READY

B16 0

| | A | B | C | D | E | F |
|---|---|---|---|---|---|---|
| 1 | | Tokimatsu Seed Liquefaction Settlement | | | | |
| 2 | | ==================================== | | | | |
| 3 | | | | | | |
| 4 | Layer | Thickness | BlowCount | Eff Shear | Vol Strain | Settlement |
| 5 | ----- | --- ft -- | --------- | --- % ---- | --- % ---- | --- In --- |
| 6 | 1 | 3.00 | 20.00 | 0.0002400 | 0.0002781 | 0.00 |
| 7 | 2 | 8.30 | 12.00 | 0.0525600 | 0.1100288 | 0.22 |
| 8 | 3 | 8.30 | 5.00 | 0.1030500 | 0.5945436 | 1.18 |
| 9 | 4 | 10.00 | 5.00 | 0.1681900 | 0.9703666 | 2.33 |
| 10 | 5 | 9.00 | 5.00 | 0.1036700 | 0.5981206 | 1.29 |
| 11 | 6 | 20.00 | 20.00 | 0.0459600 | 0.0532513 | 0.26 |
| 12 | 7 | 20.00 | 20.00 | 0.0489500 | 0.0567157 | 0.27 |
| 13 | 8 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 14 | 9 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 15 | 10 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 16 | 11 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 17 | 12 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 18 | 13 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 19 | 14 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 20 | 15 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 21 | 16 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 22 | 17 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 23 | 18 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 24 | 19 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 25 | 20 | 0.00 | 0.00 | 0.0000000 | 0.0000000 | 0.00 |
| 26 | | | | | | |
| 27 | | | | | | |
| 28 | Enter earthquake magnitude | | | 7.00 | | |
| 29 | | | | | | |
| 30 | | | | | | |
| 31 | | | | | | |
| 32 | | | Total settlement in inches | | | 4.81 |
| 33 | | | | | | |
| 34 | Enter data in columns B C and D for each sand layer | | | | | |
| 35 | Column B Layer thickness in Feet | | | | | |
| 36 | Column C Blowcounts N sub I | | | | | |
| 37 | Column D Effective Shear Strain in percent | | | | | |
| 38 | | | | | | |
| 39 | | | | | | |

What would like to do? P(rint), S(ave File), G(et File), Q(uit)

Figure 53. LIQSS results for Treasure Island using lower bound blowcounts.

```
                                                                        READY
B6   3
       A          B          C          D           E           F
1              Tokimatsu Seed Liquefaction Settlement
2              ====================================
3
4    Layer Thickness BlowCount  Eff Shear   Vol Strain  Settlement
5    ----- --- ft -- --------- --- % ----  --- % ----  --- In ---
6      1       3.00      20.00  0.0002400   0.0002781       0.00
7      2       8.30      12.00  0.0525600   0.1100288       0.22
8      3       8.30       4.00  0.1030500   0.7698489       1.53
9      4      10.00       4.00  0.1681900   1.2564860       3.02
10     5       9.00       4.00  0.1036700   0.7744807       1.67
11     6      20.00      20.00  0.0459600   0.0532513       0.26
12     7      20.00      20.00  0.0489500   0.0567157       0.27
13     8       0.00       0.00  0.0000000   0.0000000       0.00
14     9       0.00       0.00  0.0000000   0.0000000       0.00
15    10       0.00       0.00  0.0000000   0.0000000       0.00
16    11       0.00       0.00  0.0000000   0.0000000       0.00
17    12       0.00       0.00  0.0000000   0.0000000       0.00
18    13       0.00       0.00  0.0000000   0.0000000       0.00
19    14       0.00       0.00  0.0000000   0.0000000       0.00
20    15       0.00       0.00  0.0000000   0.0000000       0.00
21    16       0.00       0.00  0.0000000   0.0000000       0.00
22    17       0.00       0.00  0.0000000   0.0000000       0.00
23    18       0.00       0.00  0.0000000   0.0000000       0.00
24    19       0.00       0.00  0.0000000   0.0000000       0.00
25    20       0.00       0.00  0.0000000   0.0000000       0.00
26
27
28   Enter earthquake magnitude       7.00
29
30
31
32                  Total settlement in inches              6.04
33
34    Enter data in columns B C and D for each sand layer
35    Column B  Layer thickness in Feet
36    Column C  Blowcounts N sub I
37    Column D  Effective Shear Strain in percent
38
39
What would like to do? P(rint), S(ave File), G(et File), Q(uit)
```

Figure 54 LIQSS results for Treasure Island using average blowcounts.

```
                                                                    READY
B6   3
       A          B          C          D           E           F
1               Tokimatsu Seed Liquefaction Settlement
2               ======================================
3
4   Layer  Thickness  BlowCount   Eff Shear   Vol Strain  Settlement
5   -----  --- ft --  ---------  --- % ----  --- % ----  --- In ---
6      1       3.00      20.00   0.0002400   0.0002781        0.00
7      2       8.30      10.00   0.0525600   0.1358934        0.27
8      3       8.30       2.00   0.1030500   1.7179013        3.42
9      4      10.00       2.00   0.1681900   2.8038217        6.73
10     5       9.00       2.00   0.1036700   1.7282371        3.73
11     6      20.00      20.00   0.0459600   0.0532513        0.26
12     7      20.00      20.00   0.0489500   0.0567157        0.27
13     8       0.00       0.00   0.0000000   0.0000000        0.00
14     9       0.00       0.00   0.0000000   0.0000000        0.00
15    10       0.00       0.00   0.0000000   0.0000000        0.00
16    11       0.00       0.00   0.0000000   0.0000000        0.00
17    12       0.00       0.00   0.0000000   0.0000000        0.00
18    13       0.00       0.00   0.0000000   0.0000000        0.00
19    14       0.00       0.00   0.0000000   0.0000000        0.00
20    15       0.00       0.00   0.0000000   0.0000000        0.00
21    16       0.00       0.00   0.0000000   0.0000000        0.00
22    17       0.00       0.00   0.0000000   0.0000000        0.00
23    18       0.00       0.00   0.0000000   0.0000000        0.00
24    19       0.00       0.00   0.0000000   0.0000000        0.00
25    20       0.00       0.00   0.0000000   0.0000000        0.00
26
27
28  Enter earthquake magnitude        7.00
29
30
31
32                    Total settlement in inches              12.72
33
34   Enter data in columns B C and D for each sand layer
35   Column B  Layer thickness in Feet
36   Column C  Blowcounts N sub I
37   Column D  Effective Shear Strain in percent
38
39

What would like to do? P(rint), S(ave File), G(et File), Q(uit)
```

Figure 55. LIQSS results for Treasure Island using upper bound blowcounts.

# Appendix

## Computer Program LIQSS

### INTRODUCTION

The computer program LIQSS was developed as an interim solution for computing settlements from liquefaction of cohesionless soils. It is based on data from Reference 32 . The procedure requires a SHAKE analysis be perfromed for the site.

### SYSTEM REQUIREMENTS

The program LIQSS will operate on any PC computer with a minimum 512 k memory and DOS 3.2 or higher. Printing of results is accomplished by a parallel printer connected to LPT1 or a serial printer with LPT1 redirected.

### INPUT REQUIREMENTS

A SHAKE analysis is required for use of LIQSS program. SHAKE requires the soil deposit be divided into layers, and the design earthquake time history record be specified, and shear and damping properties of the soil layers be specified. The user is expected to be familiar with this procedure.

In addition to the SHAKE parameters, blowcounts data in the sand layers is required. Place the disk in drive A and type LIQSS to begin the program. A spread sheet will be presented. Enter data for each sand layer. Do not include nonliquefiable layers. Enter the effective cyclic shear strain from the SHAKE analysis results. As each layer is completed the settlement is computed and the total displayed. To save or print results press the **slash key** , /; then enter **P** to print, **S** to save or **G** to get a file from disk. Note the spread sheet extends from colums A through F and rows 1 through 37. Depending on the type of monitor, not all of the screen may be visible. Use the arrow keys to move about the spread sheet.

The following definitions apply:

| | |
|---|---|
| Thickness | Thickness of a layer in feet. Use same layers used in SHAKE. |
| Blowcounts | Use $N_I$ corrected blowcount for layer. |
| Effective Shear Strain | Use SHAKE data expressed as percent. |
| Magnitude | Richter magnitude of earthquake |

Computed results are:

| | |
|---|---|
| Volume Strain | Volume Strain in percent |
| Settlement | Settlement of layer in inches |
| Total Settlement | Total settlement for all liquefiable layers in inches. |

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